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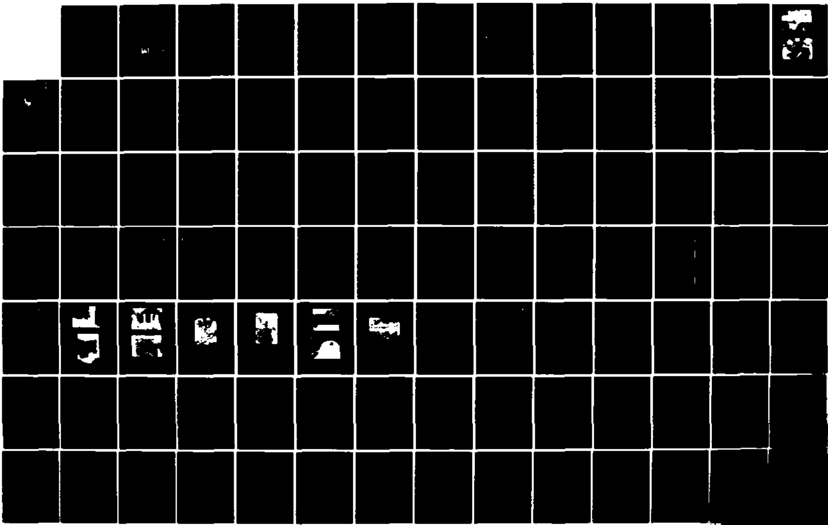

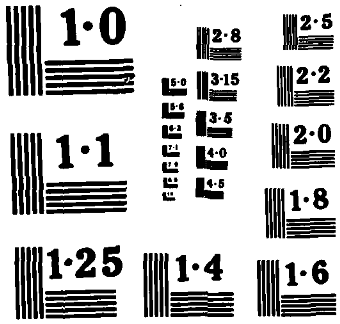

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AD-A156 431

MERRIMACK RIVER BASIN
FRANKLIN, NEW HAMPSHIRE

CHANCE BROOK DAM

NH 00410

NHWRB 87.15

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM

DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

AUGUST 1978

UNCLASSIFIED

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Franklin, New Hampshire
Chance Brook, Tributary to Pemigewasset River

20. ABSTRACT (Continue on reverse side if necessary and identify by block number):

The dam is a 14 ft. high concrete gravity structure with a 110 ft. long ogee spillway. It is intermediate in size with a high hazard potential. The spillway test flood is equivalent to the PMF. The condition of the dam is considered good, with minor repairs and additional investigatios to be made by the owner within 2 to 3 years from receipt of this report.

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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF:

NEDED

Honorable Meldrim Thomson, Jr.
Governor of the State of New Hampshire
State House
Concord, New Hampshire 03301

NOV [illegible]

Dear Governor Thomson:

I am forwarding to you a copy of the Chance Brook Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

A copy of this report has been forwarded to the Water Resources Board, the cooperating agency for the State of New Hampshire. In addition, a copy of the report has also been furnished the owner, the New Hampshire Water Resources Board, State of New Hampshire, Concord, New Hampshire 03301, ATTN: Mr. George M. McGee, Sr., Chairman.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Water Resources Board for your cooperation in carrying out this program.

Sincerely yours,

JOHN P. CHANDLER
Colonel, Corps of Engineers
Division Engineer

Incl
As stated

CHANCE BROOK DAM

NH 00410

MERRIMACK RIVER BASIN
FRANKLIN, NEW HAMPSHIRE

PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION REPORT

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NATIONAL DAM INSPECTION PROGRAM

PHASE I INSPECTION REPORT

Identification No.: NH 00410

NHWRB No.: 87.15

Name of Dam: CHANCE BROOK DAM

Town: Franklin

County and State: Merrimack County, New Hampshire

Stream: Chance Brook, Tributary to Pemigewasset River

Date of Inspection: 1 June 1978

BRIEF ASSESSMENT

Chance Brook Dam is located in Franklin, New Hampshire approximately one mile southeast of the Webster Lake outlet. The dam is a 14-foot high concrete gravity structure with a 110 foot long ogee spillway, a three-bay sluiceway with stop-logs, and a 4 foot by 4.5 foot gated sluiceway. The stop-log bays are each approximately 3-1/2 feet wide. The dam impounds water in Chance Pond and Webster Lake for recreational use. The downstream brook flows into the Pemigewasset River, which eventually discharges into the Merrimack River.

The drainage area of the dam is 19.5 square miles with rolling topography. The dam impounds a maximum of 2650 acre-feet with the pool at the top of abutments. Accordingly, the dam is classified as INTERMEDIATE in size. Its hazard classification is HIGH because of the populated area downstream of the dam. Based on size and hazard classification in accordance with Corps' guidelines, the Spillway Test Flood is equivalent to the Probable Maximum Flood (PMF).

For a dam of these characteristics, a Spillway Test Flood (STF) inflow of 24,000 cfs was selected for the entire drainage area above the dam. However, the B & M Railroad embankment about three quarters of a mile upstream of the dam restricts flow so that the STF peak discharge computed would not reach the dam. The embankment is 41 feet high with 13.6 feet x 14 feet granite arch culvert passing beneath. As long as the railroad embankment does not fail, the STF at the dam is 4000 cfs.

This flow (4000 cfs) can be discharged over the dam without overtopping assuming all stop-logs in place and the sluice gate closed. However, if the railroad embankment failed during the STF, the dam would almost certainly be overtopped, even though the spillway has a capacity of 7000 cfs.

The condition of the dam is considered GOOD, with minor repairs and additional investigations to be made by the owner within 2 to 3 years from date of receipt of the Phase I Inspection Report. Recommendations include repair of the sluice gate operating mechanism support and inspection and evaluation of the upstream railroad embankment and culvert to determine if it can retain an appropriate design flood without failure. The owner should also consider delegating a local official to open the outlet works in an emergency to decrease response time.

THE STATE OF NEW HAMPSHIRE
WILLIAM
S.
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No. 3226
REGISTERED
PROFESSIONAL ENGINEER

William S. Zoino, P.E.
N.H. Registration 3224

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STATE OF CALIFORNIA

Nicholas A. Campagna Jr., P.E.
Calif. Registration 21006

This Phase I Inspection Report on Chance Brook Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.

Charles G. Tiersch

CHARLES G. TIERSCH, Chairman
Chief, Foundation and Materials Branch
Engineering Division

Fred J. Ravens Jr.

FRED J. RAVENS, Jr., Member
Chief, Design Branch
Engineering Division

Saul Cooper

SAUL COOPER, Member
Chief, Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:

Joe B. Fryar

JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point inthe future. Only through continued care and inspection can unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test Flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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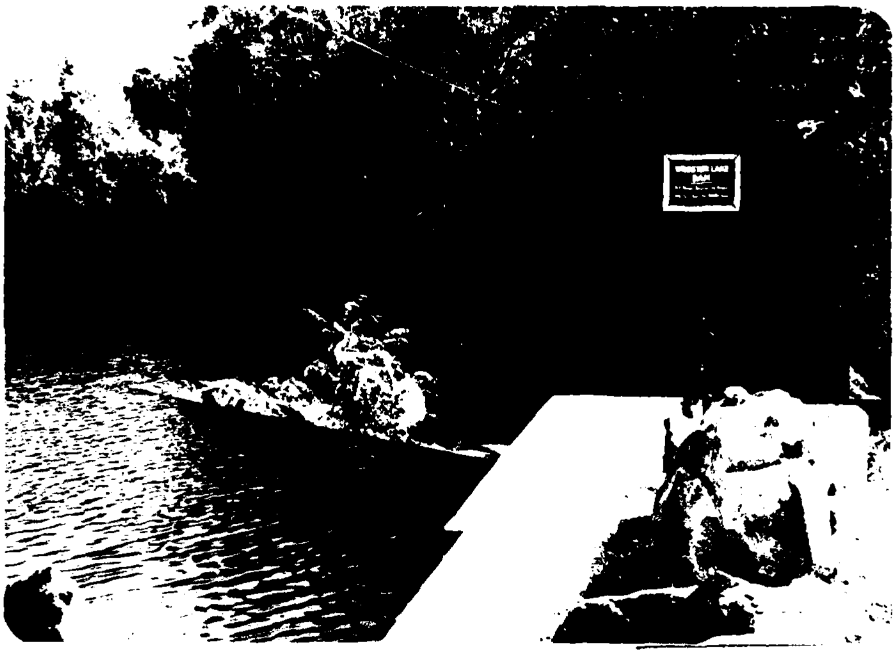


Overview from right abutment

Overview from left abutment

- SCALE -

0 1/2 1 2 miles

FROM USGS PENACOCK QUADRANGLE MAP

GOLDBERG, ZOINO, DUNNICLIFF & ASSOC, INC. GEOTECHNICAL CONSULTANTS NEWTON UPPER FALLS, MASS.	U.S. ARMY ENGINEER DIV. NEW ENGLAND CORPS OF ENGINEERS WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

LOCUS PLAN

CHANCE BROOK NEW HAMPSHIRE

FILE No. 2067

SCALE AS NOTED

DATE JULY 1978

PHASE I INSPECTION REPORT

CHANCE BROOK DAM, NH 00410

NHWRB 87.15

SECTION 1 - PROJECT INFORMATION

1.1 General

(a) Authority

Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a National Program of Dam Inspection throughout the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Goldberg, Zoino, Dunnicliff & Associates, Inc. (GZD) has been retained by the New England Division to inspect and report on selected dams in the State of New Hampshire. Authorization and notice to proceed were issued to GZD under a letter of May 3, 1978 from Ralph T. Garver, Colonel, Corps of Engineers. Contract No. DACW33-78-C-0303 has been assigned by the Corps of Engineers for this work.

(b) Purpose

(1) Perform technical inspection and evaluation of non-Federal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by non-Federal interests.

(2) Encourage and prepare the states to initiate quickly effective dam safety programs for non-Federal dams.

(3) Update, verify and complete the National Inventory of Dams.

(c) Scope

The program provides for the inspection of non-Federal dams in the high hazard potential category based upon location of the dams and those dams in the significant hazard potential category believed to represent an immediate danger based on condition of the dams.

1.2 Description of Project

(a) Location

Chance Brook Dam is located in the Merrimack River Basin on Chance Brook, approximately one mile southeast of the Webster Lake outlet and one mile west of Franklin. The locus is shown on the USGS Penacook, N.H. quadrangle. The relation of the dam to other features is shown on Figure 1 of Appendix B. Chance Brook flows into the Pemigewasset River, a tributary of the Merrimack River.

(b) Description of Dam and Appurtenances

The dam and appurtenances consist of a concrete gravity ogee type spillway structure, a concrete abutment and a gate house structure. A three bay sluiceway with stop-logs is located between the gate house structure and the right abutment. A concrete training wall is located on the right bank immediately downstream of the sluiceway. A concrete walk spans over the sluiceway. A steel handrail is located around the perimeter of the sluiceway outlet along the training wall, walkway and gate house structure. A wood frame gate house is supported on the gate house structure. All structures are founded on bedrock. Plans of this dam and appurtenant structures are not available (See overview photographs and Figures 1, 2 and 3, Appendix B for orientation).

The spillway structure is approximately 110 feet in length. This structure is laid out as an inverted vee in plan with an apex angle of approximately 110 degrees at its midpoint. The height of the spillway varies from 2 to 8 feet. The right abutment is approximately 22.5 feet long and 3.0 feet wide on its top surface. Bedrock outcrops form the left abutment.

The wood framed gate house, which is approximately 10.5 feet square, is supported on a concrete gate structure 14 feet long and 10.5 feet wide. A 5 foot square timber sluice gate is operated from within this gate house. This sluice gate controls a 4 foot wide by 4.5 foot high waterway opening. The gate is operated by a pedestal mounted hand crank. The pedestal is mounted on two steel channel sections set on the concrete floor slab.

The three bay sluiceway consists of three openings, 3.75 feet, 4.17 feet and 3.75 feet wide, respectively. Two intermediate steel stop-logs guides are set in cut-outs in the sluiceway invert and are fastened to the walkway fascia. Stop-logs have been set in place. A concrete walkway 4.17 feet wide and 15 inches thick spans over the three bay sluiceway.

(c) Size Classification

The dam impounds a maximum of 2,650 acre feet at elevation 404.2 MSL. Since the dam impounds more than 1,000 acre feet but less than 50,000 acre feet, the dam is classified as INTERMEDIATE according to the "Recommended Guidelines."

(d) Hazard Classification

The area downstream of the dam is a built-up section of the city of Franklin. Because of the potential loss of lives and extensive economic loss if the dam failed, the dam's potential hazard classification is HIGH.

(e) Ownership

The present owner of the dam is the New Hampshire Water Resources Board (NHWRB). The Board purchased the dam from Mr. George B. Horne on September 21, 1960.

Records in the files of the NHWRB indicate that in 1934 and 1938 the dam was owned by Franklin Needle Company and was used to power a sawmill on the downstream side of the structure. However, the configuration and type of dam was different than the present one. It appears that this old granite block dam was replaced by the present concrete one some time between 1938 and 1960.

(f) Operator

The operation of the dam is controlled by the New Hampshire Water Resources Board. Key officials are as follows:

George McGee, Chairman
Vernon Knowlton, Chief Engineer
Donald Rapoza, Assistant Chief Engineer
Gary Kerr, Staff Engineer

The Board's telephone number is 603-271-3406. Alternatively, the Board may be reached through the State Capital at 603-271-1110.

(g) Purpose of Dam

The dam was originally established to generate power for a mill. Since that time the dam has been reconstructed and now serves only as a recreational resource.

(h) Design and Construction History

The original granite block dam, which was constructed to generate power for a saw mill,was constructed in 1873. According to a NHWRB sketch of 1939, the granite blocks were founded on bedrock with a 4.9 foot wide gate cut into the bedrock 16 feet from the left abutment. There was also a 30 inch diameter penstock cut into the bedrock about 11 feet from the right abutment.

Sometime between 1938 and 1960 the saw mill was torn down, the penstock was removed and a concrete dam with ogee spillway was constructed. In September 1960 the NHWRB purchased the dam. In early 1973 the NHWRB constructed the three bay sluiceway with stop-logs to provide for the entire dam, sufficient discharge capacity for the 100 year storm, or 3300 cfs as determined from the Kennison-Colby formula. This is considerably more than can be passed by any of several upstream structures, as will be discussed.

(i) Normal Operation Procedures

The dam is tended by a member of the NHWRB or its staff on a week to ten day basis in the summer and on a two week interval in the winter. A log book of all visits is maintained in the Board's file.

The NHWRB maintains the stop-logs at spillway crest elevation during the summer recreational season (June 1 to mid October). If the New Hampshire Fish and Game Department notifies NHWRB that the water temperatures downstream of the dam are getting too high, NHWRB opens the gate to release cooler water.

Around mid-October the NHWRB begins pulling stop-logs to drawdown the pond level about 1-1/2 to 2 feet below the spillway crest. After periods of heavy rain the gate is opened to help maintain this drawdown level. After the winter snowmelt, the NHWRB begins replacing the stop-logs to develop a full pond at spillway crest elevation by June 1.

1.3 Pertinent Data

(a) Drainage Area

The drainage area above the dam has rolling topography with an area of 19.5 square miles. The drainage area above the B & M Railroad culvert is 17.3 square miles.

(b) Discharge at Damsite - See Stage-Discharge Curve, Appendix D.

(1) Outlet works: Gated sluiceway - 4 ft. x 4.5 ft; invert El. 392.2
Three bay sluiceway with stop-logs - 10.5 ft. wide; invert El. 393.7

(2) Maximum known flood at dam site: 1200 ± cfs (since 1960)

(3) Spillway capacity at maximum pool: 7000 cfs at El. 404.2

(4) Gated sluiceway at full pool: 135 cfs at El. 398.6 (fully open)
Sluiceway with stop-logs: 0 cfs at El. 398.6 (stop-logs in place)

(5) Gated sluiceway capacity at maximum pool: 200 cfs at El. 404.2 (fully open)
Sluiceway with stop-logs: 300 cfs at El. 40402 (stop-logs in place)

(6) Total discharge capacity at maximum pool: 7500 cfs at El. 404.2

(c) Elevation (ft. above MSL)

(1) Top of Dam: 404.2

(2) Recreation pool: 398.6

(3) Spillway crest: 398.6

(4) Invert stop-log sluice: 393.7

(5) Invert sluice gate: 392.2

(6) Streambed at downstream toe: 390

(d) Reservoir

(1) Length of maximum pool: 2.5 mi.
Chance Pond - 0.75 mi.; Webster Lake - 1.75 mi.

(2) Length of recreation pool: 2.5 mi.

(e) Storage (acre-feet)

(1) Recreation pool: 1100 at El. 398.6

(2) Top of dam: 2650 (approximately) at el. 404.2

(f) Reservoir Surface (acres)

(1) Recreation pool: 575

(2) Top of dam: 675

(g) Dam

(1) Type: Concrete gravity

(2) Length: 133 ft.

(3) Height: 14 ft.

(h) Spillway

(1) Type: Concrete Ogee

(2) Length of weir: 110 feet

(3) Crest elevation: 398.7 feet msl

(i) Regulation Outlets

The regulating outlets consist of a three bay sluiceway with stop-logs and a manually operated gated sluiceway.

The openings of the three bay sluiceway are 3.75 feet, 4.17 feet, and 3.75 feet wide, respectively with invert elevation 393.7 feet. The stop-logs in each bay are pulled and replaced by hand and lifting hook and only one or at most two boards can be so removed under head.

The gated sluiceway consists of a five foot square timber sluice gate which controls a 4 foot by 4.5 foot concrete waterway opening. The invert of the opening is elevation 392.2 feet. The gate is operated by a pedestal mounted hand crank which is in a wood framed gate house.

SECTION 2 - ENGINEERING DATA

2.1 Design

The only design data available for the present dam is a 1976 report by the NHWRB which indicates the flow capacity of Chance Brook Dam is in excess of 3300 cfs or the 100 year flood flow frequency (Kennison-Colby method). No plans or other design data of the dam could be located in the NHWRB files.

2.2 Construction

There is no known information on the construction of the dam other than the renovations which were done in 1973 by the NHWRB. In early 1973 the NHWRB's construction crew installed the three bay stop-log sluiceway.

2.3 Operation

Adequate information is available on the operation of the dam. the NHWRB has a well established schedule of visits and operational procedures. A good overall review of operation, as they relate to the unusual drainage area features, is contained in Appendix B, a Report by the NHWRB to the Governor of New Hampshire.

2.4 Evaluation of Data

(a) Availability

The prime data source is the June 1, 1978 visual inspection supplemented by conversations with staff members of the NHWRB. Definitive engineering design data are not available.

(b) Adequacy

The lack of indepth engineering data did not permit a definitive review. Therefore the adequacy of this dam could not be assessed from the standpoint of reviewing design and construction data. The evaluation is based primarily on visual inspection, past performance history, and engineering judgment.

(c) Validity

The visual inspection and hydrological analyses are of sufficient validity to permit satisfactory evaluations.

SECTION 3 - VISUAL INSPECTION

3.1 Findings

(a) General

Chance Brook Dam is in good condition at the present time. There were no findings that indicate the dam is unsafe.

(b) Dam

(1) Spillway (Photo 4)

Observations of the downstream face of the concrete spillway have revealed that there are many random, but generally minor, longitudinal cracks in the structure and one vertical crack. A horizontal crack approximately one foot below the spillway crest is prevalent over approximately 75 percent of its length. Additional horizontal cracks, varying from 5 to 30 feet in length, are prevalent throughout the downstream face of the spillway. These horizontal cracks show evidence of spalling and adjacent efflourescence, but none of the cracking is of structural significance.

A horizontal construction joint approximately 5 feet below the spillway crest was observed. This open joint starts approximately 10 feet from the gate house structure and extends in a northeasterly direction for approximately 15 feet.

A vertical crack approximately 1 inch wide at the crest and gradually tapering to a width of approximately 1/4 inch is located approximately 30 feet northeast of the gate house structure.

The upstream side of the spillway cannot be inspected due to normal water conditions.

(2) Gate House Structure (Photos 1, 2 and 5)

The inlet side of this structure has been subjected to minor erosion at the spillway crest. Fine random cracking and efflourescence is, to a limited extent, present above this erosion.

The balance of the upstream face of this structure is in good condition with the exception of minor erosion and fine random cracking in the sidewalls adjacent to the invert. A horizontal construction joint at the base of this structure adjacent to the spillway has slightly spalled and effloure sced. The balance of the downstream face of this structure and the sidewalls adjacent to the spillway are in good condition.

The wood frame gate house is in good condition.

(3) Sluice Gate

The timber sluice gate, rising stem pedestal crank mechanism are in good condition. However, the pedestal base which consists of two 8 inch by 2-1/2 inch steel channel sections is unstable. Instability is due to shearing of an anchor bolt at the end of one channel and has permitted the channel to misalign and "float." This condition has caused the pedestal apparatus to tilt out of plumb and to bind slightly at some points in the gate's travel.

(4) Sluiceway (Photo 3)

The concrete sluiceway is in good condition and does not show any evidence of checking, cracking, or spalling. The structural steel stop-log guides and the stop-logs are in good condition.

(5) Abutments

The concrete at the right abutment is in good condition without any apparent evidence of settlement or displacement. There is no evidence of cracking, checking, or spalling.

At the left abutment the concrete ogee spillway ties in to a steeply sloping bedrock outcrop. The bedrock is a massive, fine to medium grained, dark gray, micaceous, garnetiferous schist with occasional quartz seams varying from one inch to one foot thick. Jointing is irregular, but continuous high angle joints strike approximately east-west. They are spaced three to four feet apart and are tight. There is one prominent joint near the top of the left abutment that dips about 30 degrees downstream and appears to be open. At the time of June 1, 1978 inspection, water was flowing over the spillway and over the joint. However, a photo taken a month earlier when the lake level was a few inches below the spillway crest indicates there was no flow through the joint at that time.

No seepage was noted at either abutment.

(c) Appurtenant Structures

(1) Training Wall

The training wall on the right bank adjacent to the sluiceway structure is in good condition and does not show any evidence of checking, cracking, or spalling.

(2) Concrete Walkway and Steel Handrail

These supporting facilities are in excellent condition.

(d) Reservoir Area (Photos 7, 8 and 9)

The reservoir area is made up of two bodies of water, Webster Lake and Chance Pond, connected by a channel approximately 0.8 miles northwest of the dam. The channel connects the outlet of Webster Lake at the north end with Chance Pond at the south end. This channel passes through a culvert under State Route 11 and through a stone arch culvert beneath the Boston and Maine Railroad embankment.

About 0.4 miles upstream of the dam, the Carr Street embankment crosses Chance Pond. There is a 10-foot diameter corrugated steel culvert through this embankment.

The slopes along the right side of Chance Pond are stable and generally less than 6 feet high. Along the left side of Chance Pond the slopes vary from low near the left abutment of the dam to 25 to 30 feet high upstream. These slopes are relatively stable, but there is some evidence of soil creep near the toe of the steeper slope,as evidenced by the slight tilting of a few trees toward the reservoir.

(e) Downstream Channel (Photos 5 and 6)

Immediately downstream of the dam there are two channels with an island in the middle. About 100 feet downstream of the dam these channels merge into one. The channels have bedrock bottoms with occasional boulders. Some debris has collected at the toe of the spillway and the shores of the channel. However, there are no significant obstacles to flow.

The sides of the channel are relatively steep, bedrock controlled and stable.

3.2 Evaluation

The visual inspection adequately reveals key characteristics of the dam to permit satisfactory evaluation of those items which affect the stability and safety of the structure. The dam and its appurtenant works are in GOOD condition.

SECTION 4 - OPERATIONAL PROCEDURE

4.1 Procedures

As noted earlier in Section 1.2 (i), a member of the NHWRB or its staff visits this dam on a 7 to 10 day cycle during the summer and on a two week cycle during the winter. The NHWRB maintains a log book of all visits. During the summer recreational season (June 1 to mid-October) the NHWRB maintains the stop logs in the sluiceway at spillway crest elevation. If the New Hampshire Fish and Game Department determine that downstream water temperature are too high, the NHWRB opens the sluice gate to draw off cooler water from the bottom of the reservoir.

During the rest of the year the NHWRB draws down the water level about 1-1/2 to 2 feet below spillway crest elevation. This is normally controlled by pulling stop-logs, but after periods of heavy rain the sluice gate is also opened to control the water level.

After the winter snowmelt the NHWRB replaces the stop logs to bring the water level to spillway crest elevation by June 1.

4.2 Maintenance of Dam

No specific program of maintenance is currently established. The NHWRB visits the dam on a regular basis and reports any maintenance problems to the engineering section. They, in turn, assess the problem and initiate whatever corrective measures are necessary.

4.3 Maintenance at Operating Facilities

The maintenance of the operating facilities is treated in the same manner as maintenance of the dam discussed in Section 4.2 above.

4.4 Warning Systems

The NHWRB relies on its regularly scheduled site visits to detect any problems which would adversely affect dam safety. Also, after periods of heavy rain, the NHWRB schedules prompt visits to the dam to observe conditions and open discharge works as needed. The continuous interest of local residents who are quick to respond to variations in water levels also provides an informed secondary warning system. Ample evidence of this can be found in NHWRB's correspondence and phone logs.

4.5 Evaluation

In view of the characteristics of the dam and the NHWRB's regularly scheduled site visits, the operational procedures seem adequate.

SECTION 5 - HYDROLOGIC/HYDRAULIC

5.1 Evaluation of Features

(a) Design Data

The available data sources for the Chance Brook Dam include several prior inventories and inspection reports by the New Hampshire Water Resources Board (NHWRB), several letters and memoranda regarding high water levels in Webster Lake and all of the background material for flood plain mapping of Chance Brook for the Flood Insurance Study (FIS) of the Town of Franklin, New Hampshire. This last study was carried out by Anderson-Nichols Company, Inc. of Concord, New Hampshire.

Some of the basic characteristics of the dam are listed in the December 1, 1938, "Data on Dams in New Hampshire" by the New Hampshire Water Control Commission and in "Inventory of Dams in the United States" by the Corps of Engineers. In addition there exists a November 1976 report by the NHWRB relating to the high water level problems in Webster Lake.

None of these sources contains the design data for the dam, although they include some discharge calculations for both the dam and the upstream channel.

(b) Experience Data

The New Hampshire Water Resources Board has maintained daily records of water levels in Chance Pond and Webster Lake since assuming ownership in 1960. There are two gauges, one at the Chance Brook Dam with a reading of 0.0 at full lake (datum 397.7 MSL) and one at Legasee Beach on Webster Lake with a reading of 2.90 at full lake (datum 394.8 MSL). The highest stage recorded since 1960 is 2.20 feet above full lake at the gauge on the dam on April 23, 1969. This is equivalent to a flow of roughly 1200 cfs.

On July 5-6, 1973 the Legasee Beach gauge reached 3.9 but this was due to a sand bag dike that had been installed to hold the lake elevation, while at the dam the water had been drawn down to permit construction of the new sluiceway with stop-logs. At the time of the July 1973 storm, the gauge at the dam read approximately 2.5 feet.

Several complaints about the level in Webster Lake and the operation of the dam have been received by the NHWRB. However, it is the opinion of the NHWRB that these problems are caused by the natural variability in lake levels and the flow constraints between Webster Lake and Chance Pond Dam.

(c) Visual Observations

Chance Brook Dam controls the flow through Chance Brook from Webster Lake on its way to the Pemigewasset River in Franklin, New Hampshire. The structure consists of an ogee spillway about 105 feet in length, a gate house with an orifice opening roughly 4 feet by 4.5 feet and a stop-log spillway with three sets of stop-logs, each roughly 3.5 feet in width. Normal pool elevation is just below the crest of the spillway, which results in a "full lake" condition. The measurements taken during the inspection visit to the dam generally agree with the detailed survey data provided by Anderson-Nichols Company, Inc. for their FIS in Franklin.

Several constrictions in Chance Brook between Webster Lake and Chance Brook Dam were noted during the inspection visit. These constrictions could very well reduce the flow capacity in the brook and cause high water levels in Webster Lake during small floods. Only the Boston and Maine railroad bridge would severely restrict flows during a severe flood such as the Spillway Test Flood. All of the others would be overtopped and thus not significantly affect the flows reaching the dam. Basic data on these constrictions is as follows:

Carr Street: 10 ft. diameter corregated steel culvert with accumulated rocks and other debris on the invert. Road embankment allows for an 11.4 ft. depth of headwater. Invert = 393.5 MSL

R. R. Culvert: Split stone with mortar; vertical sides and arched top (Dimensions: 13.6 ft. wide x 14.0 ft. at crown). Sand, rock and debris for the stream bed. Height of embankment allows for a headwater of 38 ft. Invert = 395.0 MSL

Rte. 11: Concrete box culvert with sand, rock and collected debris for a stream bed (dimensions: 13.2 ft. wide x 8 ft. high). Headwater conditions and lakeside development allows for a maximum pond elevation of 405.7 or 10.3 ft. depth without severe damage. Invert = 394.6 MSL (in sand)

(d) Overtopping Potential

The Phase I investigation studies hydrologic conditions in order to assess the adequacy of the dam in terms of its overtopping potential and its ability to allow an appropriately large flood to pass safely. This involves comparison of a Spillway Test Flood (STF) with dam discharge and storage capacities.

The "Recommended Guidelines" of the Corps of Engineers specify procedures for determining the STF for a dam, based on its size and hazard classifications. As shown in Table 3 of the Guidelines, a dam classified as INTERMEDIATE in size with a HIGH hazard potential should have an STF equal to the Probable Maximum Flood (PMF).

The PMF is estimated using the chart of "Maximum Probable Flood Peak Flow Rates" obtained from the New England Division of the Corps of Engineers. The drainage basin above Chance Brook Dam has "rolling" topography with an area of 19.5 square miles. If the PMF is reduced slightly to account for the influence of Highland Lake, the resulting runoff rate is 1250 csm and the STF inflow discharge is about 24,000 cfs, with an assumed runoff of 19".

The B & M railroad bridge would act as a flow restriction, thus the STF peak discharge computed above would not reach the dam if the railroad embankment were not overtopped or breached. The lake must rise above 420 MSL before an alternative outlet, specifically a highway underpass 800 feet to the west of the culvert, would function.

The flow at the dam was computed by routing an assumed PMF through Webster Lake to determine the associated flow through the railroad culvert to the dam. The routing is shown in Appendix D and gives a maximum flow of 3130 cfs with the water level behind the railroad dike 23.4 feet above the invert of the culvert, or 17.9 feet above normal lake elevation.

The flow of 3130 cfs does not account for the 2.2 square miles which drain into Chance Pond between the culvert and the dam. Using the Corps of Engineers' guidelines, this area would have a PMF of 3000 to 4000 cfs, which could be expected to pass the dam before the peak outflow from Webster Lake would occur. However, since there would be some contribution from this flow at the time of peak outflow, the STF for Chance
taken as 4000 cfs. This value assumes that the railway dike will hold at a water level of 17.9 feet above normal lake elevation.

The flow from Chance Brook Dam is controlled by the 105 foot spillway, the 4 foot by 4.5 foot orifice gate and the 10.5 foot stop-log spillway. In this analysis, the stop-logs are assumed to be in place at the same level as the spillway.

In their previous FIS work, Anderson-Nichols Company, Inc. developed a rating curve assuming that the stop-logs were in place and the gate fully open (see Appendix D). According to this curve, the STF of 4000 cfs would result in a water surface elevation 3.8 feet over normal pool elevation. If the gate were closed, the maximum water level would be 3.9 feet over normal pool elevation. Neither of these levels presents any risk of overtopping the abutments, as they are at least 5.5 feet above the normal pool. The maximum discharge capacity of the dam at elevation 404.2 MSL is 7500 cfs.

Thus, as long as the railway dike which separates Webster Lake from Chance Pond holds, there seems to be little likelihood of overtopping Chance Brook Dam. However, if this dike were to fail, the dam would almost certainly be overtopped.

5.2 Hydrologic/Hydraulic Evaluation

An extrapolation of Anderson-Nichols' rating curve from their FIS work indicates that Chance Brook Dam would convey a flow of about 8000 cfs at 5.5 feet above normal pool elevation, which is the height of the dam's west abutment. If the B & M Railway Dike at the outlet of Webster Lake does not fail, this flow would not be approached under STF conditions.

However, if the dike were to fail, flow at the Chance Brook Dam could be as large as or greater than the Webster Lake peak PMF inflow of 22,000 cfs. In this case, the west abutment of Chance Brook Dam would be overtopped by several feet.

In summary, Chance Brook Dam will not be overtopped if the B & M dike holds, but it will be overtopped by several feet if the dike fails. The assessment of the adequacy of the B & M dike to withstand the STF-generated stages was beyond the scope of these Phase I investigations.

5.3 Downstream Dam Failure Hazard Estimates

The downstream flood hazards resulting from a failure of Chance Brook Dam are estimated using the procedure set forth in "Rule of Thumb Guidelines for Estimating Downstream Dam Failure Hydrographs," New England Division of the Corps of Engineers, April 1978. This procedure calls for considering the downstream attenuation of dam failure hydrographs in computing flows and flooding depths. The calculations take into account the hydraulic and storage characteristics of stream reached downstream of the dam.

For the purposes of these calculations, failure is assumed to occur at the peak water level under STF conditions, with the B & M railroad bridge intact. These conditions result in a pond level of 3.8 feet above the spillway crest or 1.7 feet below overtopping depth.

Chance Brook downstream of the dam is divided into three reaches for the analysis. The first extends from the dam to the Kimball Street Bridge, the second from the Kimball Street Bridge to the second B & M railroad bridge, and the third from the railroad bridge to the Main Street Bridge. For each reach, a typical cross-section from the FIS data was used to determine normal flow depths or the estimated peak flows. In addition, an approximate rating curve for the B & M railroad bridge at the downstream end of Reach 2 was developed since it was anticipated that Reach 2 would be subjected to the greatest potential flooding.

The analysis indicates that a depth of approximately 15 feet above the invert of the railroad bridge would result. This would be sufficient to cause moderate flood damages to structures along the south bank of the stream in Reach 2. In Reach 1, the flood depths would overflow the natural banks, but the lack of nearby structures would limit flood damages. In Reach 3, the extreme slope limits the depth of flooding, but high velocities could possibly cause severe damage to bridge abutments downstream.

It should be noted that failure of the railroad dike at the outlet of Webster Lake would, in times of high water, result in a flood flow far greater than that generated by failure of Chance Brook Dam alone. Indeed, the dam's failure would become incidental due to the volume of water already released from the dike.

While it is beyond the scope of these Phase I investigations to study the structural soundness and hydraulic implications of the B & M railroad dike, this is an important area for further study.

SECTION 6 - STRUCTURAL STABILITY

6.1 Evaluation of Structural Stability

(a) Visual Observations

There are no design data available for review of the structural stability of the dam and appurtenant structures. The extensive field investigations and findings do not indicate any displacement and/or distress which would warrant the preparation of structural stability calculations based on assumed sectional properties and technical values.

Observations during the inspection period have revealed various minor deficiencies which can be attributed to alternate freeze and thaw cycles resulting in spalling and cracking of concrete and minor maintenance required on the sluice gate crank pedestal support.

(1) Spillway

The horizontal joints in the spillway structure can be attributed to lack of quality control in placement of concrete. These joints are randomly dispersed througout the structure and do not pose any factor detrimental to the stability of the structure. The vertical crack in the spillway is the result of a faulty construction joint which has been subjected to minor cavitation.

(2) Gate House Structure

Minor erosion with resulting fine random cracking and efflourescence of concrete is the result of alternate freeze and thaw cycles due to the near constant water surface elevation in the lake during the winter.

(3) Sluice Gate Support Assembly

The shearing of an anchor bolt on this assembly suggests previous problems with vertical alignment of the sluice gate. The prudent operation of the sluice gate, as demonstrated by a representative of the NHWRB, can circumvent any operational problems provided that experienced personnel maintain the facility.

(b) Design and Construction Data

According to the "Inventory of Dams in the U.S.A." dated 12 March 1974, the dam was originally completed in 1873. Subsequent to this date the entire original structure, including an adjacent mill were removed. Historical records indicate that the original dam, a stone structure, was in existence in 1938. Additional research has revealed that the present day dam was constructed prior to 1960. Furthermore the gate house structure was constructed subsequent to 1938 based on materials and construction technology. The three bay sluiceway was constructed by the NHWRB in 1973 to provide additional discharge capacity. Further design and construction data are not available.

(c) Operating Records

The NHWRB has good records since its assumption of ownership in 1960.

(d) Post Construction Changes

Since the present day concrete dam was constructed sometime prior to 1960, the only noted change was the addition of the three-bay stop-log sluiceway which was constructed by the NHWRB in 1973.

(e) Seismic Stability

The dam is located in Seismic Zone No. 2 and in accordance with recommended Phase I guidelines does not warrant seismic analyses.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS & REMEDIAL MEASURES

7.1 Dam Assessment

(a) Condition

The visual inspection revealed no deficiencies of major concern. The dam is in GOOD condition. There is adequate spillway capacity to pass the Spillway Test Flood provided the B & M railroad embankment at the outlet of Webster Lake does not fail.

(b) Adequacy of Information

The information available is adequate as a basis on which to form evaluations.

(c) Urgency

The sluice gate operating mechanism support should be repaired by the owner, and the railroad embankment should be investigated within the next two to three years after receipt of the Phase I Investigation Report.

(d) Need for Additional Information

Available information indicates no necessity for additional information at this time, other than supplementary studies recommended below.

7.2 Recommendations

During large floods the upstream B & M railroad embankment with limited size culvert acts as a dam. An investigation and evaluation of this embankment should be made to determine if it is capable of retaining an appropriate design flood without failure and to field check the elevation data taken from the 5 foot contour map which indicates that no other outlet to the lake is available below elevation 420 MSL.

7.3 Remedial Measures

(a) Alternatives

An alternative to evaluating the railroad embankment's performance as a dam might appear to be the provision of sufficient discharge capacity beneath the embankment to prevent detention during an appropriate design flood. However, the viability of the alternate is lessened by the then necessary provision of added discharge capacity at the Chance Brook Dam.

(b) O & M Procedures

Without the services of a skilled operator knowledgeable in the operating characteristics of the sluice gate, this gate conceivably can be subject to failure. Due to the lack of restraint of the operating mechanism support it is recommended that the owner repair this to allow sluice gate operation without unusual stress or binding to the timber gate.

To decrease the response time in opening the outlet works in an emergency, the NHWRB should consider delegating some operational responsibility to a local official such as the police or fire chief. This individual would maintain a set of keys to the gate house with instructions on removing stop-logs and operating the sluice gate in an emergency, as directed by the NHWRB.

Removal of all debris from the immediate downstream channel will insure unimpeded flow.

APPENDIX A

VISUAL INSPECTION CHECKLIST

INSPECTION TEAM ORGANIZATION

Date: 1 June 1978, 8:45 a.m.

Project: Chance Brook Dam, NH 00410
Franklin, New Hampshire
Chance Pond Brook
NHWRB 87.15

Weather: Sunny, warm, moderate breeze

Inspection Team

James H. Reynolds	Goldberg, Zoino, Dunnicliff & Associates, Inc. (GZD)	Team Captain
William S. Zoino	GZD	Soils
John E. Ayres	GZD	Geology
Nicholas A. Campagna	GZD	Soils
Andrew Christo	Andrew Christo Engineers, Inc. (ACE)	Structural & Concrete
Paul Razgha	ACE	Structural & Mechanical
Guillermo Vicens	Resource Analysis, Inc.	Hydrology

State Official Present

Ken Stern, New Hampshire Water Resources Board

Chance Brook Dam
Franklin, N.H.

June 1, 1978
NH 00410

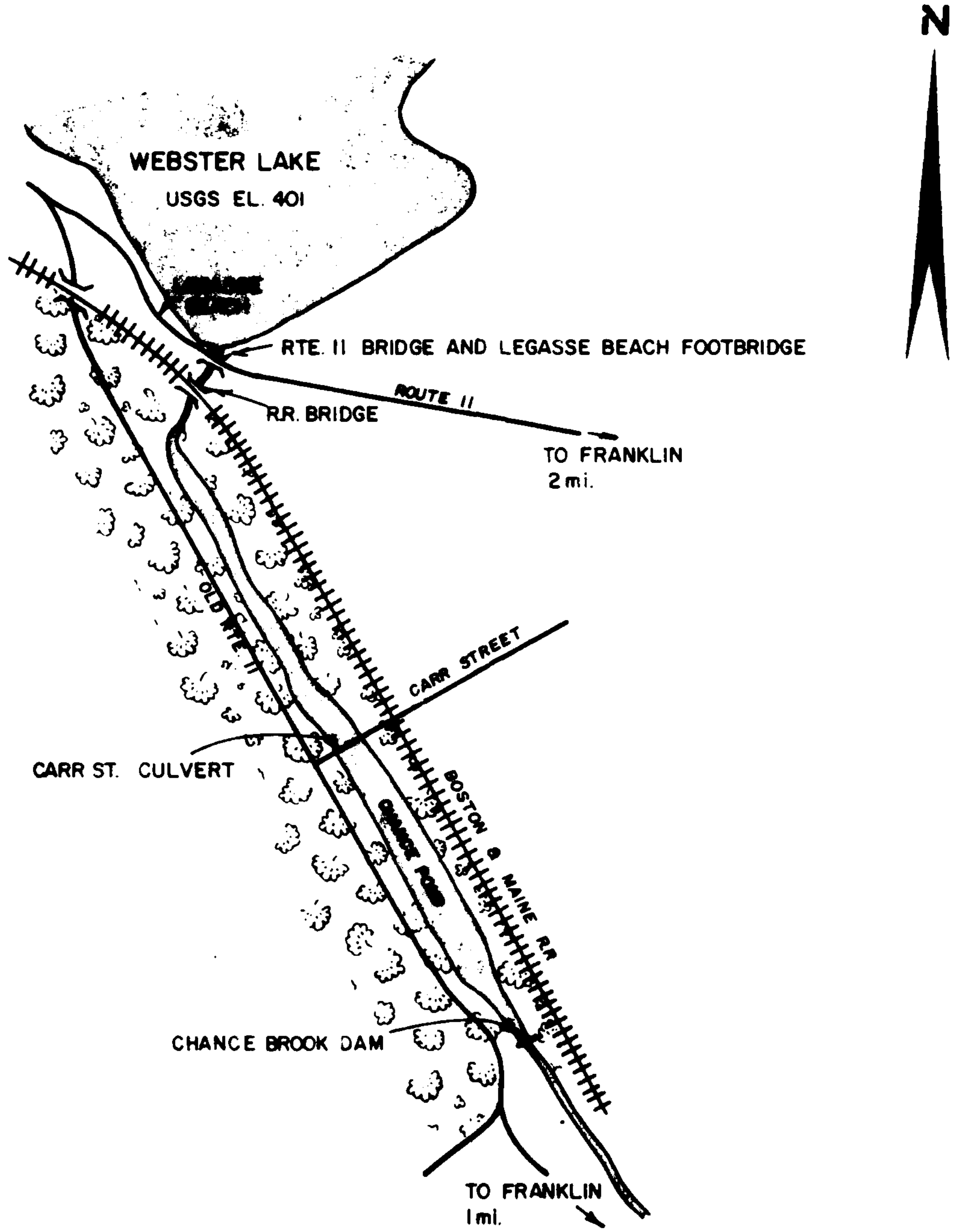
CHECK LISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
Dam Superstructure		
Vertical Alignment	nac	Good
Horizontal Alignment		Good
Settlement		None
Lateral Movement		None
Downstream Seepage		None
Concrete		Good
Foundation Drainage Features	nac	None
Outlet Works		
Spillway	AC	Random horizontal open joints with evidence of spalling and adjacent efflourescence. One vertical crack.
Sluice Gate Inlet	AC	Minor erosion at spillway crest elevation, fine random cracking and efflourescence.
Sluice Gate Outlet	AC	Minor erosion and fine random cracking in sidewall adjacent to invert. Horizontal joint, slight spalling and efflourescence.
Sluice Gate	AC	Timber gate and operations mechanism in good condition. Pedestal base unstable and must be repaired.
Sluiceway	AC	Good
Stop-logs and supports	AC	Good

CHECK LISTS FOR VISUAL INSPECTION

AREA EVALUATED	BY	CONDITION & REMARKS
Abutments		
Right abutment concrete	nc	Good
Seepage	nac	None
Left abutment	nac	Massive bedrock with tight joints
Appurtenant Structures		
Training Wall	nc	Good
Wood Frame Gatehouse	nc	Good
Concrete walkway and steel handrail	nc	Good
Reservoir		
Shoreline	nac	Generally stable, minor soil creep at toe of steep slopes on left shoreline, evidenced by slight tilting of trees toward reservoir, 500 to 600 feet upstream of dam.
Upstream hazard in the event of backflooding	nac	Numerous shore front houses subject to inundation if water rises more than 3 feet above spillway crest.
Downstream Channel		
Debris	nac	Numerous logs and branches
Trees overhanging channel	nac	None
Obstructions	nac	No major obstructions, occasional boulders.

Appendix B



GOLDBERG, ZOINO, DUNNICLIFF & ASSOC., INC.
GEOTECHNICAL CONSULTANTS
NEWTON UPPER FALLS, MASS.

U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

FIG. 1

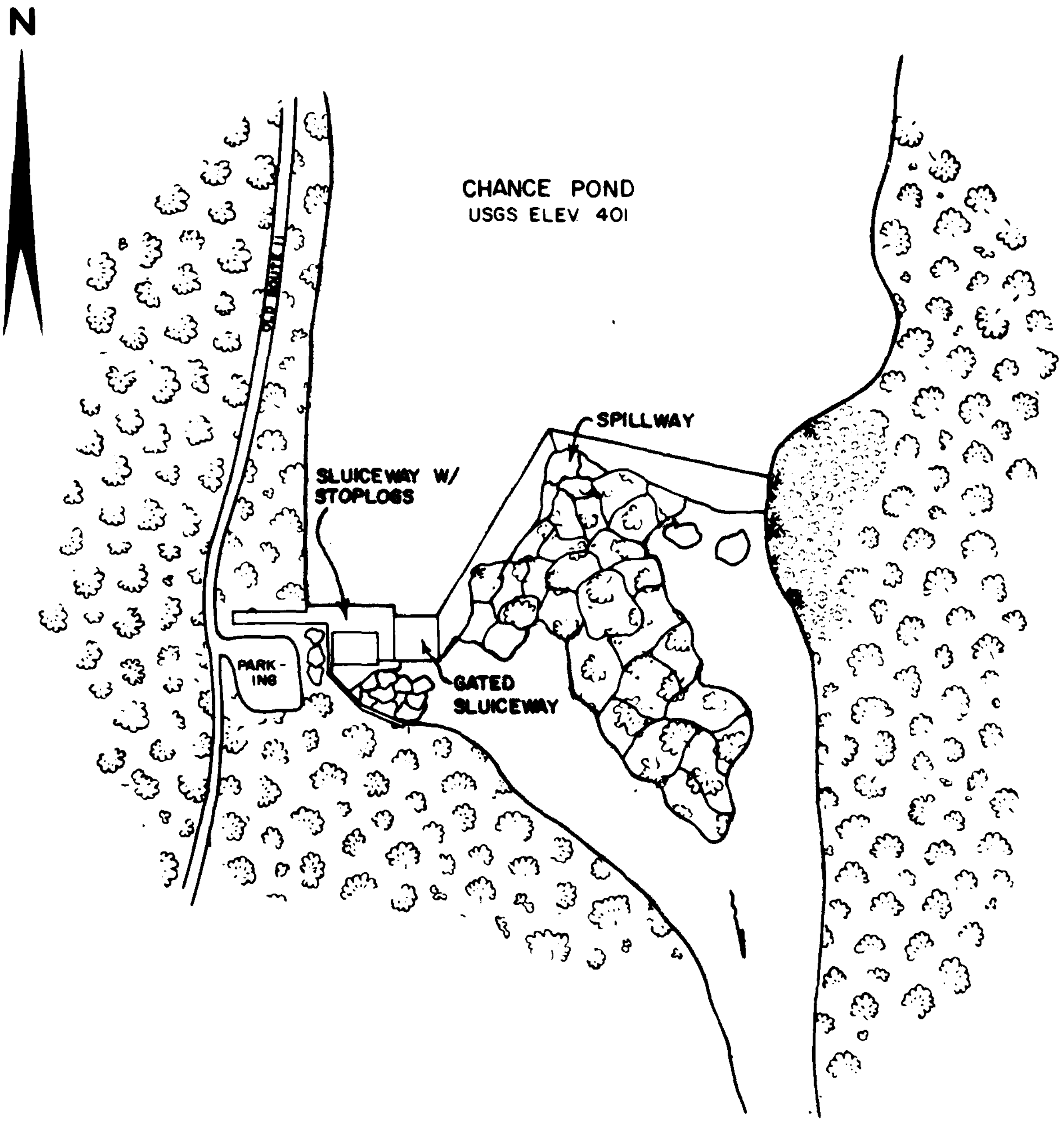
SITE PLAN I

CHANCE BROOK NEW HAMPSHIRE

SCALE NO SCALE

DATE JULY 1978

FILE No. 2067



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CORPS OF ENGINEERS
WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED DAMS

FIG. 2

PLAN OF DAM

CHANCE BROOK — NEW HAMPSHIRE

FILE No. 2067

SCALE 1" = 40'
DATE JULY 1978

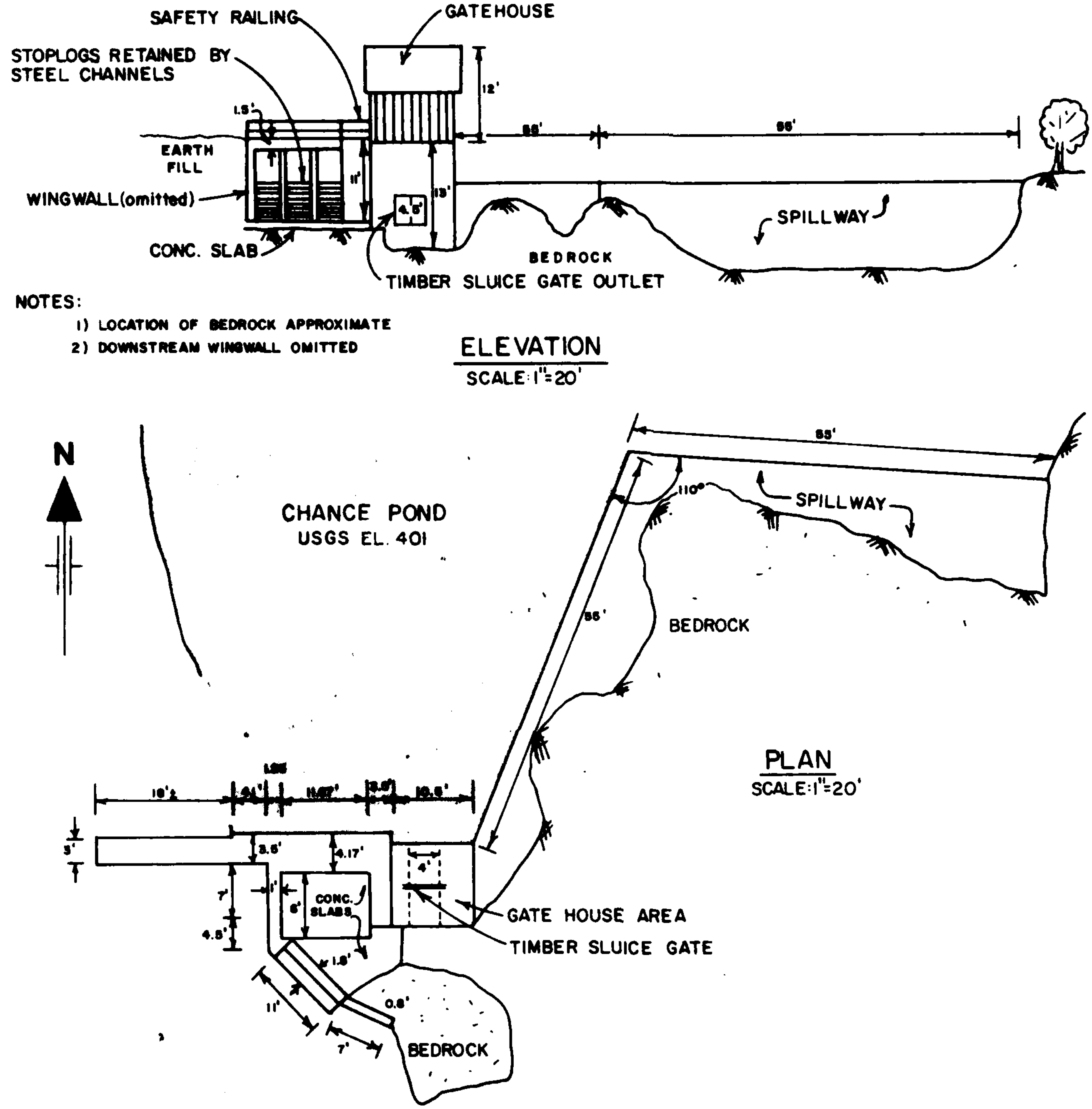

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FIG. 3

PLAN AND ELEVATION OF DAM

CHANCE BROOK — NEW HAMPSHIRE

FILE No. 2067

SCALE AS NOTED

DATE JULY 1978

The following records are maintained by the NHWRB at their Concord offices:

(1) A letter dated 6 May 76 from the Mayor of Franklin, NH to the NHWRB concern the control of Webster Lake.

(2) A letter dated 25 Jul 63 from an unknown agency to the Public Utilities Commission concerning the lake level.

(3) An undated report from 1961 by the NHWRB concerning their investigation of a high water complaint at the lake.

The Board can be reached at 603-271-3406 or through 603-271-1110.

REVIEW OF WEBSTER LAKE OPERATION

FOR

1) Governor's Office

2) Mayor of Franklin, New Hampshire

BY

New Hampshire Water Resources Board

November 1976

WEBSTER LAKE AND CHANCE POND

HYDRAULICS REVIEWED

INTRODUCTION

A stretch of pondage approximately 4500 feet beginning at the outlet of Webster Lake and ending at the Water Resources Board's dam has been studied many times for various reasons over the past years. This review has been prompted by correspondence from the Governor's office and from the Mayor's office of Franklin. They relate to complaints of spring-time "high water" and mismanagement of the lake and Chance Pond Brook. Within this stretch there are five man-made structures, each with different hydraulic characteristics and flow capacities, across the brook. Chance Pond, as controlled by the dam, is a flooded portion of the brook which backs up to the outlet of Webster Lake and thus effects the level of the lake. See accompanying photos and map.

During the course of this study, a variety of information sources were tapped with the bulk coming from the Board's record files. The remaining portion from interviews of local residents and information from personnel of the City Managers office of Franklin.

BACKGROUND INFORMATION

This Board's records include information dated in the early 1930s to the present concerning the lake and the dam, part of which is a diary of pond levels from 1970 to the present. This report contains several hydrographs of the gages at the dam and Webster Lake.

The purpose of the hydrograph is to indicate how the pond stage fluctuates with time. Pond stage, or elevation, is a measure of the amount of the water in storage; which is directly affected by inflow from the surrounding area and the outflow through the dam or stream channel. If inflow exceeds outflow, then the stage increases and vise versa. Also included on the hydrographs is the sequence of operations at the dam. As can be seen from the graphs, when an operation is accomplished the two pond stages are affected in either rise or fall of the pond surface.

An interpretation of these hydrographs brings to light several ideas which are discussed below. These charts indicate the following basic data and were chosen for being representative of the era with the most complete available gage readings.

1. U.S.G.S. Gage readings at Legasse Beach
2. Gage readings at the Webster Lake dam
3. The relative positions of the gate and stoplogs at the dam as it is operated.

Each hydrograph legend is self-explanatory. The first and probably most

obvious is that during a given calendar year the summer months exhibit a reasonably stable pond elevation while late Fall, Winter and early Spring are marked with fluctuations. This is caused by the Fall and Winter drawdown operation, and Spring snow melt and rains. It also indicates that operations at the dam such as opening the gate or pulling stoplogs rapidly drains Chance Pond, but that Webster Lake is lowered more slowly.

When the pond levels are fairly stable there appears to be a differential of 0.1 ft. to 0.2 ft. in the gage heights. Since the difference is small and steady, it is probably due to the two gages having a different datum. The slight variations are probably caused by misreading or recording of the gage. Even with this constant error, valid assumptions and recommendations can be made to take corrective action.

The single, most important observation is that whenever an operation, such as pulling stop logs or opening the gate, is accomplished, the pond at the dam is lowered significantly more than Webster Lake. It should also be noted that the converse is true. This situation is due to upstream conditions limiting the inflow to Chance Pond and thereby restricting the overall flow capacity of the dam.

BASIC FACTS

Starting at the Lake and working downstream the restricting man-made structures are as listed below:

1. Foot bridge near U.S.G.S. gage at Legasee Beach
2. Rte. #11 Bridge
3. Boston & Maine Railroad Bridge
4. Carr Street corregated metal bridge
5. Webster Lake dam

In addition to these there are three major natural conditions which cause a varying effect on the stream flow. These are:

1. A constantly changing outlet elevation of Webster Lake.
2. The ever-changing swamp conditions that exist between the railroad bridge and the open water of Chance Pond.
3. The sand from Legasse Beach as it drifts through the Rte. 11 and railroad bridges, and into the swamp. It also eventually drifts all the way down to the dam and silts in Chance Pond.

The flow capacities of these various conditions as outlined above see tabulated here for easy reference.

Webster Lake Dam	In excess of 3300 cfs or the 100 yr. flood flow frequency (Kennison-Colby method)

Carr Street	800 cfs without flow over road and backwater elevation to 404.76 (Design Capacity)
Natural Channel (thru x-section)	945 cfs @ elev. 405.0
R. R. Culvert	1294 cfs @ elev. 405.0
Rt. 11	800 cfs to F.G. (w/sand as stream bed)
	900 cfs to F.G. (w/invert cleaned to concrete slab)

CHANNEL AND BASIC STRUCTURE DETAILS

Dam: 105 ft. long concrete ogee spillway; 4 ft. x 4 ft. gate and 3 - 4 ft. long stop log bays.

Carr Street: 10 ft. diameter corregated steel culvert with accumulated rocks and other debris on the invert. Road enbankment allows for an 11.4 ft. depth of headwater.

Natural Channel @ X-section: Double channel with sand and rock stream bed and bushes growing on the sides of the stream banks.

R. R. Culvert: Split stone with mortar; vertical sides and arched top (Dimensions: 13.6 ft. wide x 14.0 ft. at crown). Sand, rock and debris for the stream bed. Height of enbankment allows for a headwater of 38 ft. However, upstream development severly limits available headwater.

Rte. 11: Concrete box culvert with sand, rock and collected debris for a stream bed (dimensions: 13.2 ft. wide x 8 ft. high). Headwater conditions and lakeside development allows for a maximum pond elevation of 405.7 or 10.3 ft. depth without severe damage.

U.S.G.S. Gage reads 2.90 at full pond and would read 8.00 at 405.7.

DISCUSSION

In reviewing the structures it becomes self-evident that the dam has the greatest flood flow capacity and the Carr Street culvert the least from a design standpoint. The actual flow at Carr Street is reduced below design figures due to the presence of debris, rocks, and silt that are partially blocking it. However, the Rt. 11 bridge is a close second due to the lakeside development limiting the available headwater depth. The only structure which could take the flow of a

100-year storm without damage to the surrounding area is the dam at Chance Pond. The backwater from the dam would not innundate Carr Street. However, the culvert has such a limited capacity that flow over the road would occur. This would also cause a backwater effect through all of the other structures and thus raise the level of Webster Lake. It has been estimated by the Baker Engineers of Harrisburg, Pa., for a HUD preliminary flood study, that at flood stage Webster Lake would be 6.8' above full pond. This translates to a gage reading of 9.7' or elev. of 407.8. Unfortunately many cottages, septic systems, wells, etc. are located below the 405.0' elev.; so the amount of damage and flooding would be serious.

Two isolated complaints of "high water" have been filed and reviewed during the course of this study. One is related to the bridge over Sucker Brook on Lake Shore Drive. Field investigations indicate an active beaver dam downstream of the bridge that causes a pond to form under the bridge and thus reduces the overall flow capacity of the brook and structure. The remedy would be to remove the beaver and dam.

The other complaint involves a house recently built along Rt. 11 and in close proximity to a seasonal stream. The basic complaint deals with the high lake levels of the June - July 1973 storm. The owner, Mr. Whiting says that the Lake backs up into the culvert and floods the area when the gage approaches 3.6 and causes damage at stage 4.0' plus. His preference would be 0.9' and 1.9' gage height for winter and summer respectively.

The primary reason for this report is to respond to the Webster Lake Association and the Mayor of Franklin regarding the charge of mismanagement of the dam at Webster Lake. The various groups and individuals around the lake, and the State statutes governing the management and operation of State controlled dams dictate that an exceptionally small tolerance in the fluctuation of the Lake and pond is to be required. This tolerance of 1.5 inches from the anticipated pond level for a stipulated time is just not feasible. Given the restrictive flow conditions upstream of the dam and the hydrological aspects of the drainage area, the tight control of the Lake and pond surfaces is not possible even with continuous operations at the dam. The hydrographs substantiate this situation. For example, one inch of runoff from the entire drainage area has the potential for raising the lake nearly 30 inches, or 2.5 feet! Using the Baker Engineers' estimate for the 100-year flood the Lake would rise 6.8 feet. If this depth were added to a "full pond", then the surface would be at elev. 407.8 feet (9.7' on gage). The dam could be wide open and this would still occur. Even if the Lake were down to the natural conditions, that is with the streambed controlling the elevation of the pond, a 6.8 foot rise would mean a flood stage of 403.3 or 5.1 on the beach gage. It should be quite obvious from the preceding discussion that even a moderate rain storm adversely affects the Lake levels regardless of any human influences. Then compounding the situation by constructing restrictive bridges and a dam to maintain an artifically high pond level only worsens the situation. As people encroach upon the Lake and stipulate demands not physically possible the net result becomes damage to the environment and the surrounding buildings.

It has been suggested that the winter lake level be drawn down 1' or 2' from "full pond" and stabilized there until the springtime whereupon "full lake" would again be strickly maintained. This type of operation is fairly common and possible except that a stabilized pond level for the Webster Lake drainage system due to its hydrologic nature is not possible. In areas where the Lake or pond is spring-fed with only minor runoff (no surface streams) contributing to the stored water, a stable pond elevation is possible. Webster Lake, however, does not meet this criteria. Due to the numerous streams, swamps and other lakes all contributing to Webster Lake the water surface will fluctuate uncontrollably. With the restrictions as indicated previously an even higher than"normal" lake level develops. This situation occurs for each and every storm regardless of the duration or so-called frequency of reoccurence.

In a review of the outlet channel under Route 11 several assumptions were made:

1. Dam open full and Chance Pond drained.

2. Carr Street did not exist.

3. Natural channel restrictions eliminated downstream of the railroad bridge.

This allows the outlet channel (w/beach) to control the lake level and would be considered as natural as possible with the existing lakeside development. With a pond stage of 3.0 on the gage (0.1' above "full pond") the outflow would be 20[illegible] cfs. If that level were lower by 2.1 feet (0.9' on gage), then a mere 80-85 cfs would be flowing. If a 15-year storm were to hit the drainage area, then a peak flow of 750 to 800 cfs would be entering the Route 11 bridge area.

Since this far exceeds the low flow stage, the Lake would naturally rise until the flow through the bridge area equaled 750-800 cfs (estimated to be elev. 405'). So even without the dam or Carr Street bridge a natural rise of pond elev. would occur for each storm with flows that exceed the discharge through the system prior to the storm. This being the case fluctuations in lake surface elev. must be acknowledged and anticipated.

CONCLUSIONS & RECOMMENDATIONS

The drainage for Webster Lake is a "wet" system with many small ponds and swamps, and streams all contributing a flow of water to the Lake. Since nature, at best, is only somewhat predictable, management of this natural resource can be difficult and at times even hazardous. Due to the hydraulics and hydrological conditions, controlling the lake level via the dam at Chance Pond to within a 1.5 inch tolerance is simply not a feasible demand. Two feet to 2.5 feet is more realistic and history proves that point.

There are three primary problem areas, namely:

1. Carr Street culvert is significantly undersized for the volume of water it must carry.

2. Rt. 11 bridge flow capacity is reduced from optimum by sand and debris silting in the channel from the lake outlet through the "swamp" to Chance Pond.

3. Lakeside development is encroaching upon the shoreline and significantly reducing the areas over which the lake previously flowed without causing damage.

The remedies are simple - remove the problems.

1. The Carr Street road enbankment and entire culvertshould be removed completely to eliminate the primary restriction of storm flood flows. An alternative would be to increase the size of the culvert to pass the 100-year storm. This would be on the order of a bridge with a waterway opening 10' high by 30' wide.

2. The silt and debris that exists in the outlet channel should be removed to re-establish the previously determined elev. of the stream bed at Legasse Beach. This would also require a better control of the movement of sand from the beach exists today. The stream gradient from the lake to the dam should be of uniform slope, and this can be accomplished by dredging and removing the accumulated silt.

3. The operation of the dam could be modified to:

   a) Extend the drawdown period to include the spring snow melt and high runoff and fill the Lake after this time period has passed.

   b) Increase the drawdown from one foot to three or four feet to accumulate snow melt and spring runoff in storage.

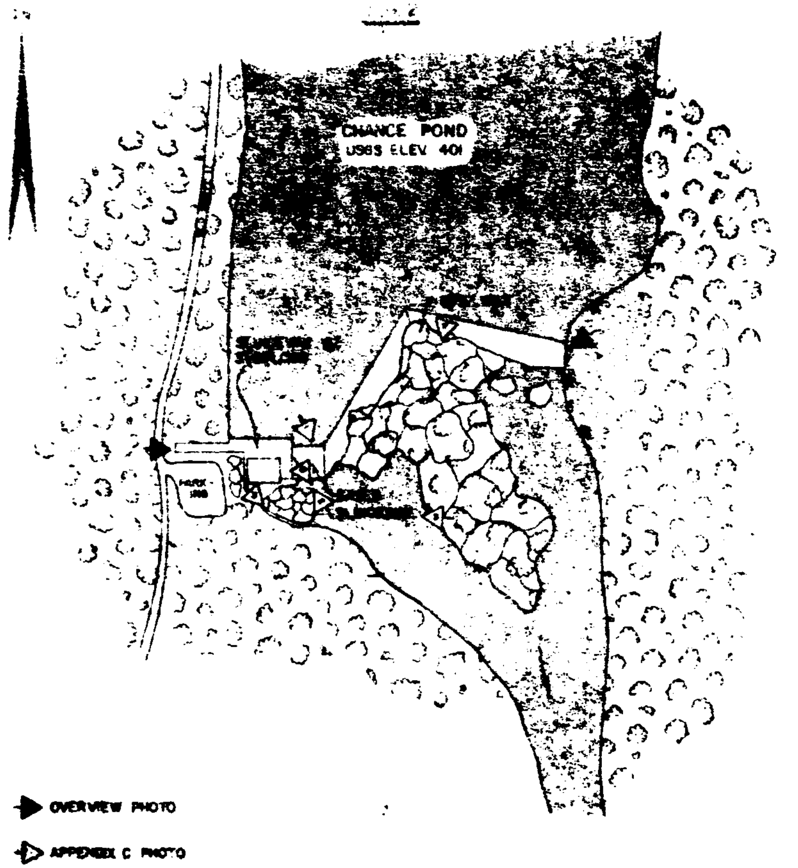
   c) Incorporate both 3a + 3b into one operation. This is the much preferred operational remedy to the problem.

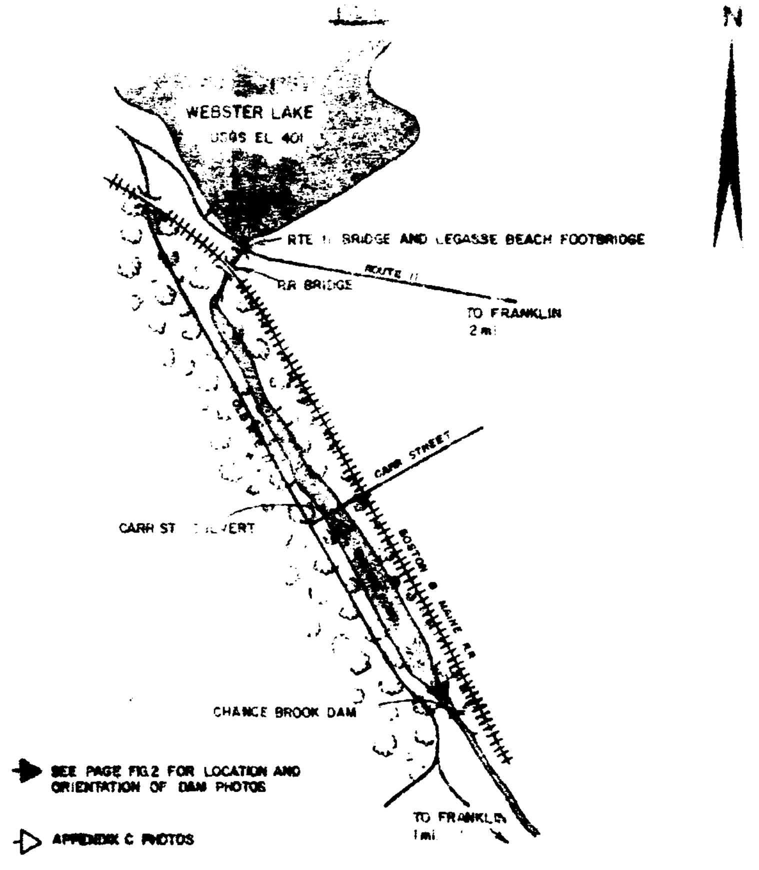
This in effect would be returning the lake to its "natural"conditions in the winter months and allowing the sand bar at the beach to control the level on Webster Lake.

4. The problem of the existing lakeside development does not have an acceptable simple solution. Due to the nature and location of these encroachments upon the shoreline, the inhabitants must acknowledge the anticipated fluctuations of the lake and enjoy all of the consequences.

APPENDIX C

SELECTED PHOTOGRAPHS



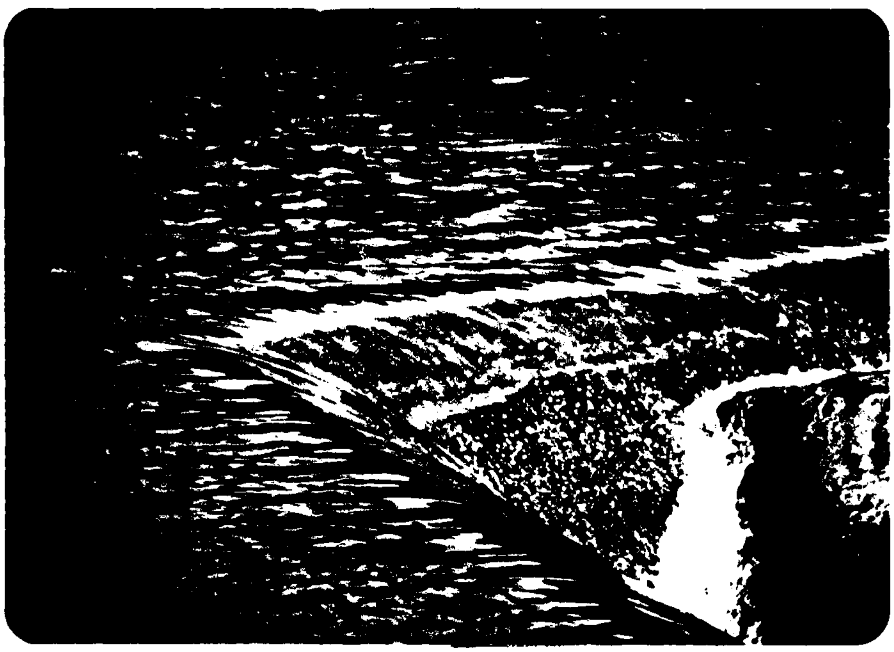


LOCATION AND ORIENTATION OF PHOTOS

1. Detail of gatehouse inlet from upstream

2. Detail of gatehouse outlet from downstream right side

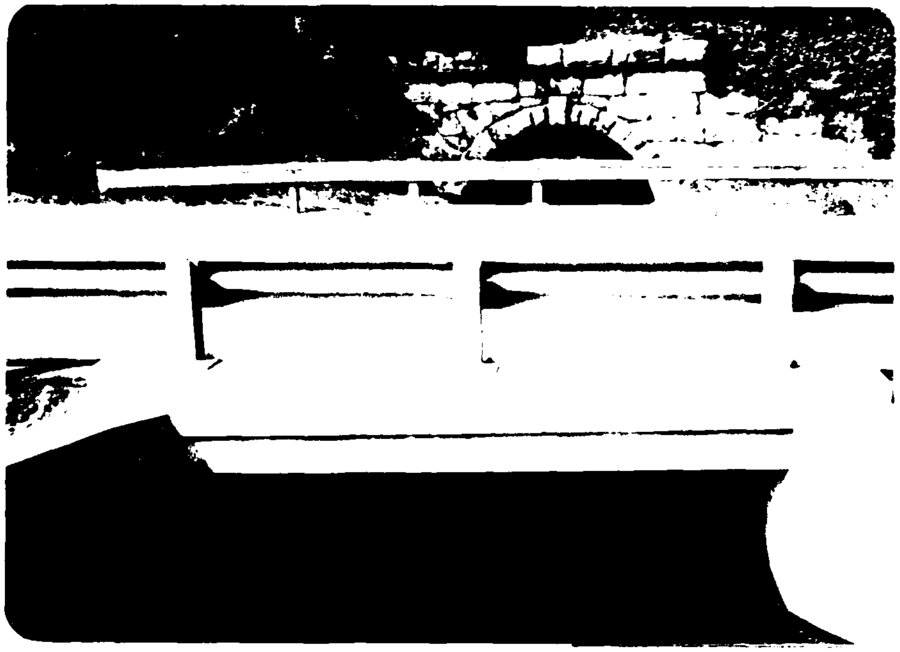
3. Sluiceway outlet from downstream right side



4. Detail of spillway crest from downstream

5. View of gatehouse and debris from downstream channel

6. View of downstream channel from gatehouse



7. View from upstream of road bridge and railroad bridge near lake outlet

8. View from upstream of railroad culvert near lake outlet

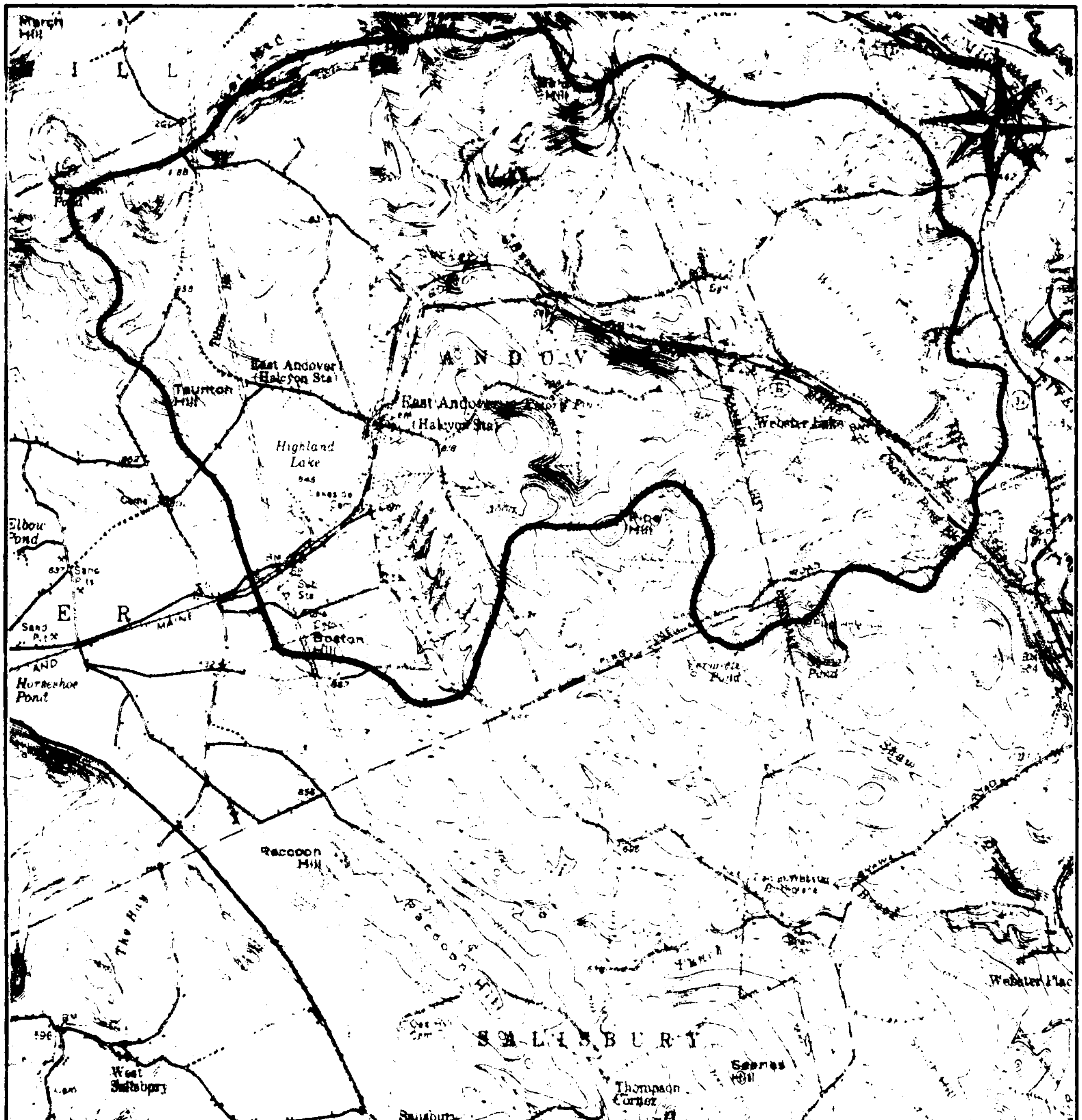
9. View from downstream of Carr St. culvert

APPENDIX D

HYDROLOGIC AND HYDRAULIC COMPUTATIONS

FOR

CHANCE BROOK DAM



--SCALE--

0 1/2 1 2 miles

FROM: USGS PENACOOK, N.H. QUADRANGLE MAP

FILE No. 2067

GOLDBERG, ZOINO, DUNNICLIFF & ASSOC, INC.
GEOTECHNICAL CONSULTANTS
NEWTON UPPER FALLS, MASS.

U.S. ARMY ENGINEER DIV. NEW ENGLAND
CORPS OF ENGINEERS
WALTHAM, MASS.

NATIONAL PROGRAM OF INSPECTION OF NON-FED. DAMS

DRAINAGE AREA

CHANCE BROOK — NEW HAMPSHIRE

SCALE AS SHOWN

DATE JULY 1978

THE CHANCE POND DAM IS USED TO CONTROL WEBSTER LAKE AS WELL AS CHANCE POND. BASED ON THE IMPOUNDMENT VOLUME OF WEBSTER LAKE IT IS CLASSIFIED AS AN INTERMEDIATE SIZE. THE LOCATION OF THE BUILT UP SECTION OF FRANKLIN DOWNSTREAM OF THE DAM REQUIRES A HAZARD CLASSIFICATION OF HIGH.

GIVEN AN INTERMEDIATE SIZE AND HIGH HAZARD THE SDF = PMF.
FROM THE C.O.E CURVE FOR A DRAINAGE AREA OF 19.5 SQMI THE RUNOFF RATE ≈ 1250 CSM IF WE CREDIT HIGHLAND LAKE WITH SOME INFLUENCE SO THAT A VALUE BELOW THE "ROLLING" LINE IS APPROPRIATE. THE RESULTING SDF ≈ 24000 CFS.

BUT — BOTH THE ANDERSON-NICHOLS WORK ON THE HUD F.I.S. AND A NHWRB REPORT TO THE MAYOR OF FRANKLIN, NOV. '76, DISCUSS THE FACT THAT THERE IS SIGNIFICANT RESTRICTIONS TO FLOW AT THE OUTLET OF WEBSTER LAKE, UPSTREAM OF THE DAM.

ANCO FOUND THAT ROUTING THE Q_{10}, Q_{50}, Q_{100}, AND Q_{500} INFLOWS THROUGH WEBSTER LAKE RECOGNIZING THE CONSTRICTIONS AT THE OUTFLOW RESULTED IN VERY MUCH REDUCED OUTFLOWS.

A SIMILAR ROUTING WAS NECESSARY FOR THE PMF. BASED ON THE 1250 CSM RATE AND A REDUCED AREA OF 17.3 SQ MI FEEDING WEBSTER LAKE THE PEAK INFLOW WAS ESTIMATED AS: $17.3(1250) = 21625 \approx 22000$ CFS.

FOR A ROUTING IT IS NECESSARY TO ASSIGN A HYDROGRAPH SHAPE. ANCO (SEE ATTACHED SHEET) ESTIMATED A TIME OF CONCENTRATION OF 12.2 HRS. BUT FOR THE RISING LIMB OF THE HYDROGRAPH 11 HOURS WAS SELECTED. WE ACCEPTED THE 11 HOURS AS AN ~~ACCEPTABLE~~ REASONABLE APPROXIMATION.

THE COE CONSIDERS THE MAXIMUM PROBABLE RUNOFF TO EQUAL 19". THUS IT WAS NECESSARY TO DEVELOPE A HYDROGRAPH WITH A PEAK OF 22000 CFS AT 11 HOURS AND A TOTAL VOLUME EQUAL TO A 19" RUNOFF. ATTACHED IS THE ADOPTED HYDROGRAPH OF INFLOW TO WEBSTER LAKE.

Anderson-Nichols & Company, Inc.

JOB NO. 2789 36

Subject: Webster Lake

Routing

Sheet No ___ of ___

Date 9/23/77

Computed ___

Checked ___

SQUARES 1/4 IN. SCALE 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30

"Representative"

Determination of t_c (time of concentration)

Divide path of flow into five reaches to account for slope changes & both overland flow and flow through lakes (Highland & Webster)

Use equation $t_c = \dfrac{L^{1.15}}{7700\, H^{0.38}}$

Kirpich

t_c = time of concentration; hrs
L = length of reach; feet
H = elevation difference between highest & lowest points in feet

Reach 1: from divide near Taunton Hill to swampy area at contour crossing of 660'.
Length = 1.34 miles; Height = 1020 - 660 ft

t_c = .371 hrs

Reach 2 from 660 contour to Highland Lake
Length = 1.35 miles; Height = 660 - 645 ft

t_c = 1.252 hrs

Reach 3: across Highland Lake
Length = 0.9 miles, Height = .20 ft (24")

t_c = 4.05 hrs.

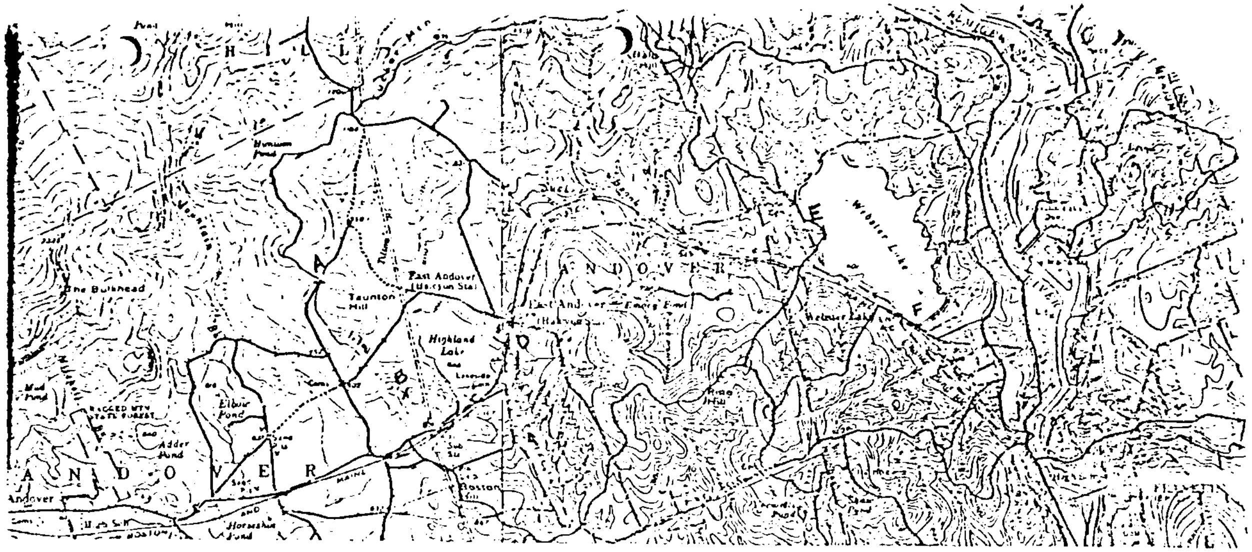
Reach 4 from Highland Lake to Webster Lake through Sucker Brook
Length = 4.85 miles; Height = 645 - 401

t_c = 1.88 hrs.

Reach 5 from inlet to outlet of Webster Lake
Length = 1.10 miles, Height = .25 ft (3")

t_c = 4.69 hrs.

Total time of concentration = 12.21 hrs.



	Length	Height
REACH 1 (A→B)	1.54 mi	1020-660
REACH 2 (B→C)	1.35 mi	660-645
REACH 3 (C→D)	0.7 mi	
REACH 4 (D→E)	4.35 mi	645-401
REACH 5 (E→F)	1.1 mi	

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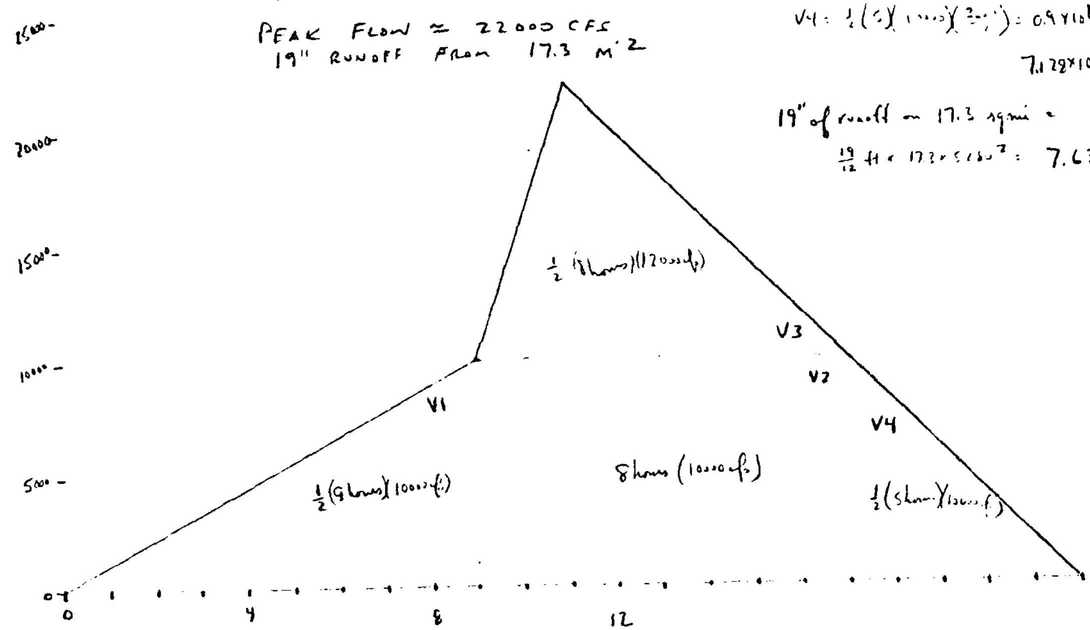
INFLOW HYDROGRAPH
TO WEBSTER LAKE
PEAK FLOW ≈ 22000 CFS
19" RUNOFF FROM 17.3 MI 2



V1 = ½(9)(10000)(3600/1) = 1.62×10⁸ ft³
V2 = 8(10000)(3600/1) = 2.88×10⁸ ft³
V3 = ½(8)(12000)(3600/1) = 1.728×10⁸ ft³
V4 = ½(5)(10000)(3600/1) = 0.9×10⁸ ft³
7.128×10⁸ ft³

19" of runoff on 17.3 sq mi =
$\frac{19}{12}$ ft × 17.3 × 5280² = 7.636×10⁸ ft³

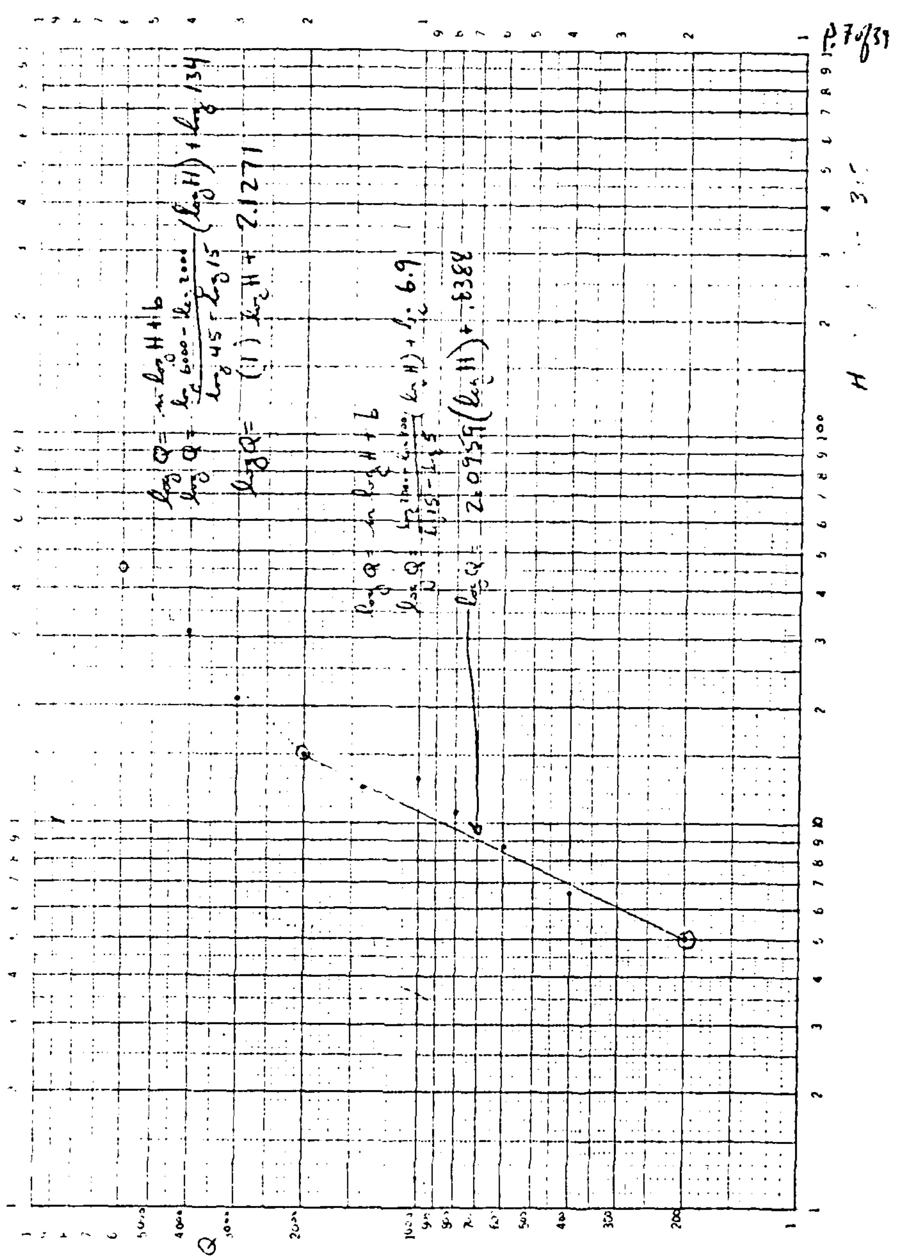
DAMS 148 | CHANCE POND #9 WEBSTER LAKE | 7-11-79 DWW p. 6 of 31

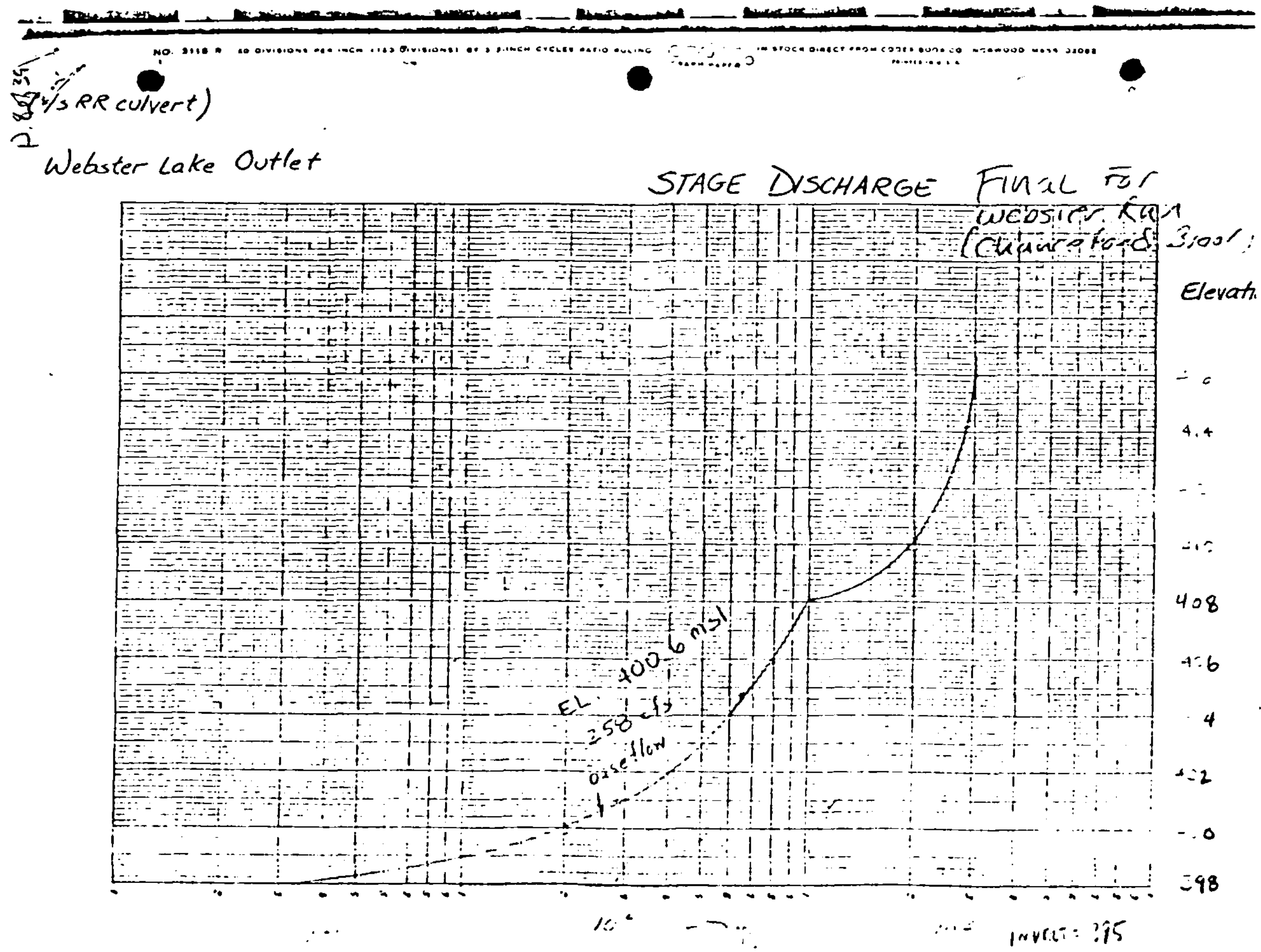
TO SET THE DISCHARGE-STAGE RELATIONSHIP WE UTILIZED A RATING CURVE DEVELOPED BY ANCO USING HEC-2 AND BACKWATERING THROUGH THE RAILROAD CULVERT, THE ASSUMPTION IS MADE THAT AT THE VERY HIGH FLOW LEVELS THE B+M RAILROAD CULVERT BECOMES THE CRITICAL CONTROL SECTION, THE ATTACHED ANCO COMPUTER OUTPUT FOR THE SECOND SECTION # 1.63 WAS USED REPRESENTATIVE STAGES REFERENCED TO A CULVERT INVERT OF 395.0 MSL. THESE WERE PLOTTED ON LOG-LOG PAPER AND TWO "LINEAR" APPROXIMATIONS DETERMINED,

WHERE H IS DISTANCE ABOVE ELEV. 395

FOR $H < 15$ $\log Q = 2.0959(\log H) + .8388$

FOR $H \geq 15$ $\log Q = \log H + 2.1271$


K·E
46 7320
Q
H
$\log Q = n \log H + b$
$\log Q = (1)\log_e H + 2.1271$


(u/s RR culvert)
Webster Lake Outlet
STAGE DISCHARGE Final for
Webster Run
Elevation
4.4
408
4
398
EL 400.6 msl
258 cfs
INVERT 395

P.9939

SECTION NUMBER	CHANNEL LENGTH	MIN EL OF ROADWAY	MAX EL OF LOW CHORD	MIN EL GROUND	DISCHARGE (CFS)	CWSEL	CRIWS	EG	TOPWID	DEPTH
1.59	496.30	0.0	0.0	396.70	200.00	399.90	0.0	399.94	53.23	3.20
1.59	496.00	0.0	0.0	396.70	400.00	401.43	0.0	401.51	275.38	4.76
1.59	496.00	0.0	0.0	396.70	600.00	403.27	0.0	403.28	267.88	6.57
1.59	496.00	0.0	0.0	396.70	800.00	405.20	0.0	405.21	302.10	8.50
1.59	496.00	0.0	0.0	396.70	1000.00	407.26	0.0	407.26	324.73	10.56
1.59	496.00	0.0	0.0	396.70	1400.00	405.44	0.0	405.47	304.86	8.74
1.59	496.00	0.0	0.0	396.70	2000.00	406.29	0.0	406.33	314.23	9.59
1.59	496.00	0.0	0.0	396.70	3000.00	407.33	0.0	407.38	325.62	10.63
1.59	496.00	0.0	0.0	396.70	4000.00	408.17	0.0	408.25	334.88	11.47
1.61	84.00	0.0	0.0	394.20	200.00	399.92	0.0	400.02	14.00	5.72
1.61	84.00	0.0	0.0	394.20	400.00	401.41	0.0	401.65	14.00	7.21
1.61	84.00	0.0	0.0	394.20	600.00	403.11	0.0	403.47	14.00	8.92
1.61	84.00	0.0	0.0	394.20	800.00	405.00	0.0	405.43	14.00	10.80
1.61	84.00	0.0	0.0	394.20	1000.00	407.03	0.0	407.51	14.00	12.83
1.61	84.00	0.0	0.0	394.20	1400.00	404.79	0.0	406.18	14.00	10.60
1.61	84.00	0.0	0.0	394.20	2000.00	405.00	0.0	407.71	14.00	10.60
1.61	84.00	0.0	0.0	394.20	3000.00	405.41	405.41	411.08	14.00	11.21
1.61	84.00	0.0	0.0	394.20	4000.00	407.84	407.84	414.65	14.00	13.64
1.63	90.00	436.00	406.70	395.00	200.00	399.97	0.0	400.10	14.00	4.97
1.63	90.00	436.00	406.70	395.00	400.00	401.52	0.0	401.83	14.00	6.52
1.63	90.00	436.00	406.70	395.00	600.00	403.24	0.0	403.70	14.00	8.24
1.63	90.00	436.00	406.70	395.00	800.00	405.14	0.0	405.70	14.00	10.14
1.63	90.00	436.00	406.70	395.00	1000.00	407.13	0.0	407.90	14.00	12.13
1.63	90.00	436.00	406.70	395.00	1400.00	405.30	0.0	406.99	14.00	10.30
1.63	90.00	436.00	406.70	395.00	2000.00	407.29	0.0	409.39	14.00	12.29
1.63	90.00	436.00	406.70	395.00	3000.00	413.12	0.0	415.29	14.00	18.12
1.63	90.00	436.00	406.70	395.00	4000.00	423.88	0.0	425.40	14.00	28.88
1.63	21.00	0.0	0.0	394.40	200.00	400.01	0.0	400.12	13.40	5.61
1.63	21.00	0.0	0.0	394.40	400.00	401.61	0.0	401.88	13.40	7.21
1.63	21.00	0.0	0.0	394.40	600.00	403.70	0.0	403.82	269.83	9.30
1.63	21.00	0.0	0.0	394.40	800.00	405.84	0.0	405.87	468.79	11.44
1.63	21.00	0.0	0.0	394.40	1000.00	408.12	0.0	408.13	573.39	13.72
1.63	21.00	0.0	0.0	394.40	1400.00	407.47	0.0	407.50	543.63	13.07
1.63	21.00	0.0	0.0	394.40	2000.00	410.00	0.0	410.02	660.08	15.60
1.63	21.00	0.0	0.0	394.40	3000.00	415.94	0.0	415.94	1000.62	21.54
1.63	21.00	0.0	0.0	394.40	4000.00	425.85	0.0	425.85	2098.77	31.45

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[illegible]	4000.000	425.664	[illegible]	0.0	0.0	2090.000	[illegible]892
1.641	200.000	400.139	0.0	0.002	0.0	20.108	0.0
1.641	400.000	401.902	1.764	0.001	0.0	20.143	-0.035
1.641	600.000	404.073	2.171	0.059	0.0	20.186	-0.077
1.641	800.000	405.835	1.762	0.0	0.0	399.009	-378.900
1.641	1000.000	405.170	2.335	0.024	0.0	564.689	-544.581
1.641	1400.000	407.510	-0.660	0.009	0.0	517.415	-497.306
1.641	2000.000	410.101	2.590	0.046	0.0	715.919	-695.811
1.641	3000.000	416.035	5.934	0.048	0.0	1013.219	-993.110
1.641	4000.000	425.914	9.880	0.030	0.0	2090.000	-2069.892
1.644	200.000	400.142	0.0	0.003	0.0	38.344	0.0
1.644	400.000	401.956	1.814	0.053	0.0	93.395	-55.051
1.644	600.000	404.202	2.247	0.129	0.0	282.795	-244.450
1.644	800.000	405.927	1.724	0.092	0.0	413.450	-375.106
1.644	1000.000	403.130	2.254	0.010	0.0	567.775	-529.431
1.644	1400.000	407.552	-0.629	0.041	0.0	524.654	-486.310
1.644	2000.000	410.104	2.553	0.004	0.0	716.278	-677.934
1.644	3000.000	416.034	5.929	-0.001	0.0	1025.308	-986.963
1.644	4000.000	425.914	9.880	-0.000	0.0	2090.000	-2051.656

DATA FOR LAST CROSS SECTION

PROFILE	TYPE ENC	TARGET	TOP WIDTH AREA-ACRES	TOP WIDTH AREA-DIFF
1	0.0	0.0	18.730	0.0
2	0.0	0.0	21.731	3.001
3	0.0	0.0	23.165	4.435
4	0.0	0.0	25.101	6.371
5	0.0	0.0	26.612	7.882
6	0.0	0.0	25.975	7.245
7	0.0	0.0	27.015	8.285
8	0.0	0.0	28.540	9.809
9	0.0	0.0	31.347	12.617

FOR THE STORAGE - STAGE RELATIONSHIP THE ANCO COMPUTATIONS OF AREAS AT ELEVATIONS 400, 405, 410, AND 415 WERE USED, AND THEN THE INCREMENTAL STORAGE ABOVE ELEV. 399 PLOTTED. FROM THIS TWO FUNCTIONS WERE DERIVED WHERE H IS IN FEET ABOVE 395 SEE ATTACHED SHEETS.

$H = (S/666.67) + 4 \qquad S < 8000$

$H = ((S - 8000)/750) + 12.4 \qquad S \geq 8000$

WHERE S = STORAGE ABOVE ELEV 399

H = HEAD ABOVE ELEV 395

...son-Nichols & Company, Inc.

JOB NO.

Subject ______

Sheet No. ______ of p. 12 of 39
Date ______
Computed ______
Checked ______

SQUARES TO IN. SCALE 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30

[illegible] Lake

Elevation/Depth	Area	Acres	Volume	
399 [illegible]	[illegible]	678.—		
			691 acre feet	
400/1 [illegible]	1.0325	661.—		
			3451	[illegible] 4.45
[illegible]	[illegible]	720.—		
			3922	7117
[illegible]	1.2075	769.—		
			3919	
[illegible]	[illegible]	823.—		

To determine volume assume frustum of pyramid

$V_{volume} = \frac{1}{3} h (B_1 + B_2 + \sqrt{B_1 B_2})$

[illegible] 399'–400'

$V = \frac{1}{3}(1')(\text{[illegible]}) = 691$ acre feet

[illegible] 400'–405'

$\frac{1}{3}(5')(\text{[illegible]}\, 661 + \sqrt{661 \cdot 720}) = 3451$ acre feet

[illegible] 405'–410'

$\frac{1}{3}(5')(769 + 720 + \sqrt{720 \cdot 769}) = 3922$ acre feet

[illegible] 410'–415'

$V = \frac{1}{3}(5')(\text{[illegible]} + 823 + \text{[illegible]}) = 3919$ acre feet

2789-36
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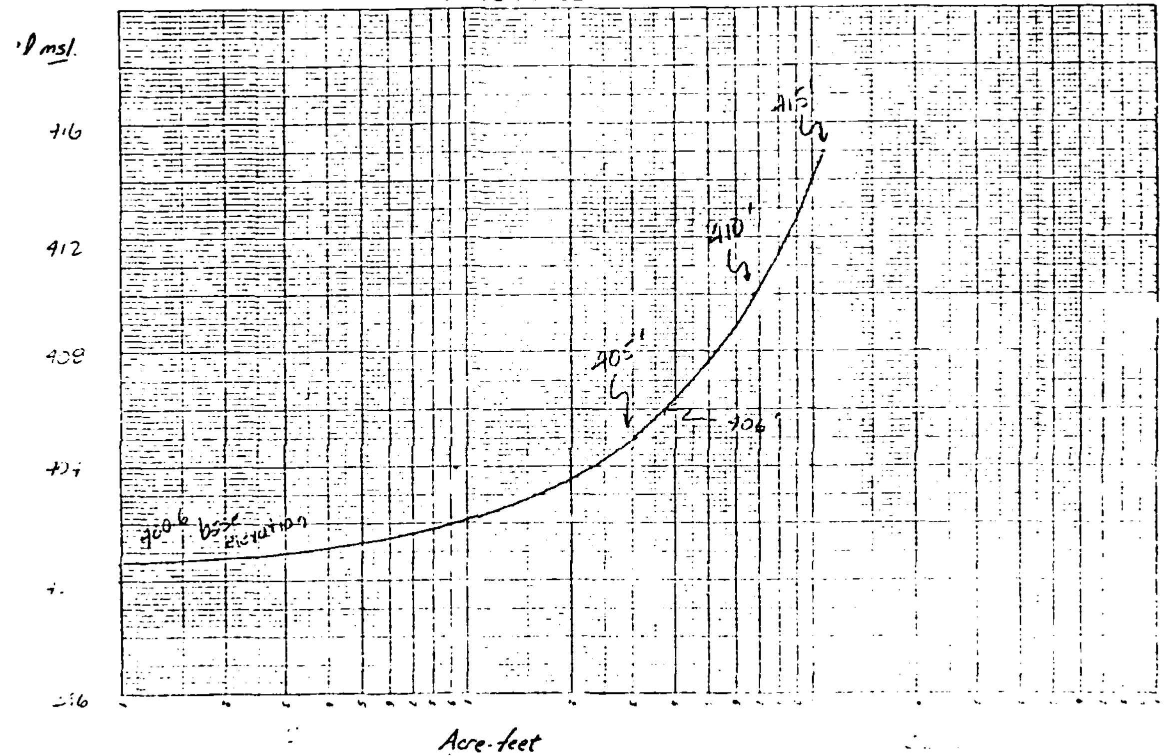
SURCHARGE STORAGE WEBSTER LAKE



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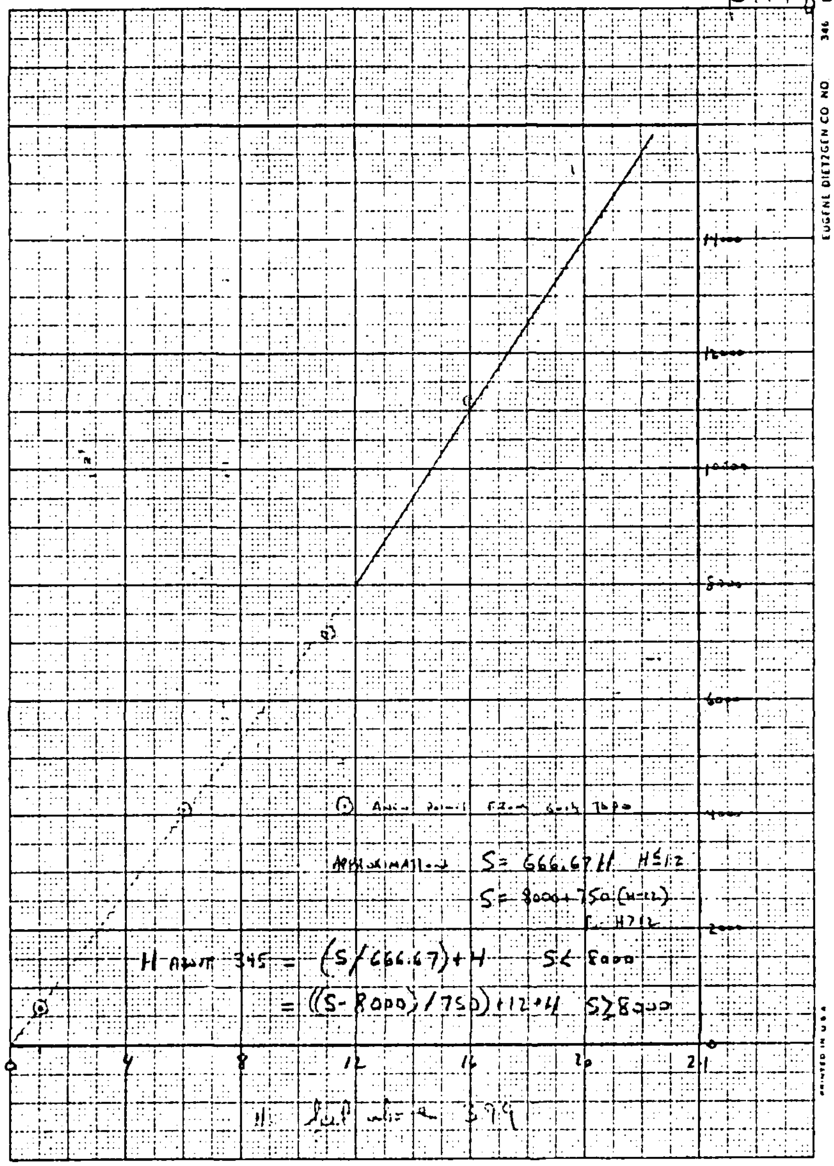
14000
12000
10000
8000
6000
4000
2000
0
4
8
12
16
20
24
S= 666.67 H H≤12
S= 8000+750(H-12)
H>12
= ((S-8000)/750)+12+4 S≥8000
EUGENE DIETZGEN CO NO 346

DAMS 148 | CHANCE POND #9 | 7-13-79 DWW | p. 15 of 39

WEBSTER LAKE

FOR THE ROUTING OF THE PMF INFLOW THROUGH WEBSTER LAKE THE FOLLOWING FORMULATION WAS USED.

① ASSIGN INITIAL VALUES

H_o = LAKE STAGE AT START OF STORM
V_o = VOLUME = $f(H_o)$
Q_o = OUTFLOW = $f(H_o)$
I_o = INFLOW

H_o WAS IN FEET ABOVE RR CULVERT INVERT $H_o = 5.5'$

V_o = VOLUME ABOVE 399 FROM STORAGE CURVE = 1000 AF

Q_o = DISCHARGE FROM CURVE ≈ 250 CFS

I_o = INITIAL INFLOW SET = 0.

② USING A 10 MIN TIME STEP, FOR EACH t.

V_t = VOLUME AT END OF TIME STEP
I_t = INFLOW AT END OF TIME STEP
H_t = AVG DEPTH DURING TIME STEP
Q_t = AVG OUTFLOW DURING TIME STEP

$$V_t = V_{t-1} + \left(\left(I_t + I_{t-1}\right)/2\right) + Q_{t-1}\right)\Delta t$$

$$H_t = f\left(\frac{V_t + V_{t-1}}{2}\right)$$

$$Q_t = f(H_t) = g\left(\frac{V_t + V_{t-1}}{2}\right)$$

THE ROUTING WAS PROGRAMED IN BASIC ON THE TECHTRONIX COMPUTER AND RUN FOR THE 22 PERIOD OF THE ASSUMED INFLOW HYDROGRAPH.

A LISTING OF THE PROGRAM AND OUTPUT ARE ATTACHED.

```
LIST
100 REM PROGRAM TO ROUTE PMF THROUGH WEBSTER LAKE
110 REM JOB 148 DAM SAFETY
120 V=1000*43560
130 Q=250
140 I=0
150 FOR K=1 TO 132
160 IF K=1 THEN 220
170 IF K=31 THEN 220
180 IF K=61 THEN 220
190 IF K=91 THEN 220
200 IF K=121 THEN 220
210 GO TO 280
220 PAGE
230 PRINT "ROUTING OF PMF THROUGH WEBSTER LAKE"
240 PRINT USING 250:
250 IMAGE // 5T"TIME"11T"VOLUME"20T"INFLOW"28T"DISCHARGE"42T"HEAD"
260 PRINT USING 270:
270 IMAGE 5T"HRS"12T"AC-FT"22T"CFS"31T"CFS"42T"FEET"
280 IF K>54 THEN 310
290 I1=K*185.185193
300 GO TO 350
310 IF K>66 THEN 340
320 I1=10000+(K-54)*1000
330 GO TO 350
340 I1=22000-(K-66)*333.33333
350 I2=(I+I1)/2
360 V1=V+I2*10*60-Q*10*60
370 I=I1
380 V2=(V1+V)/2
390 V3=V2/43560
400 V=V1
410 IF V3>8000 THEN 440
420 H=V3/666.666667+4
430 GO TO 450
```

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```
440 H=(V3-8000)/750+12+4
450 IF H<15 THEN 480
460 L=LGT(H)+2.1271
470 GO TO 490
480 L=2.0959*LGT(H)+0.8388
490 Q1=10↑L
500 T=K*10/60
510 PRINT USING 520:T,V3,I2,Q,H
520 IMAGE 4D.2D,8D,9D,9D,9D.2D
530 Q=Q1
540 NEXT K
550 END
```

ROUTING OF PMF THROUGH WEBSTER LAKE

TIME HRS	VOLUME AC-FT	INFLOW CFS	DISCHARGE CFS	HEAD FEET
0.17	999	93	250	5.50
0.33	998	278	246	5.50
0.50	1000	463	245	5.50
0.67	1004	648	246	5.51
0.83	1011	833	246	5.52
1.00	1020	1019	247	5.53
1.17	1032	1204	249	5.55
1.33	1047	1389	250	5.57
1.50	1063	1574	252	5.60
1.67	1083	1759	255	5.62
1.83	1105	1944	258	5.66
2.00	1129	2130	261	5.69
2.17	1156	2315	264	5.73
2.33	1186	2500	268	5.78
2.50	1218	2685	273	5.83
2.67	1252	2870	277	5.88
2.83	1289	3056	283	5.93
3.00	1329	3241	288	5.99
3.17	1371	3426	294	6.06
3.33	1415	3611	301	6.12
3.50	1462	3796	308	6.19
3.67	1511	3981	315	6.27
3.83	1563	4167	323	6.34
4.00	1617	4352	332	6.43
4.17	1674	4537	340	6.51
4.33	1733	4722	350	6.60
4.50	1794	4907	360	6.69
4.67	1858	5093	371	6.79
4.83	1924	5278	382	6.89
5.00	1993	5463	394	6.99

ROUTING OF PMF THROUGH WEBSTER LAKE

TIME HRS	VOLUME AC-FT	INFLOW CFS	DISCHARGE CFS	HEAD FEET
5.17	2064	5648	406	7.10
5.33	2137	5833	419	7.21
5.50	2213	6019	433	7.32
5.67	2291	6204	447	7.44
5.83	2371	6389	462	7.56
6.00	2454	6574	478	7.68
6.17	2539	6759	495	7.81
6.33	2627	6944	512	7.94
6.50	2717	7130	531	8.07
6.67	2809	7315	550	8.21
6.83	2903	7500	569	8.35
7.00	3000	7685	590	8.50
7.17	3098	7870	612	8.65
7.33	3199	8056	635	8.80
7.50	3303	8241	658	8.95
7.67	3408	8426	683	9.11
7.83	3516	8611	708	9.27
8.00	3626	8796	735	9.44
8.17	3738	8981	762	9.61
8.33	3852	9167	791	9.78
8.50	3969	9352	821	9.95
8.67	4087	9537	852	10.13
8.83	4208	9722	884	10.31
9.00	4331	9907	918	10.50
9.17	4459	10500	952	10.69
9.33	4597	11500	989	10.90
9.50	4748	12500	1030	11.12
9.67	4913	13500	1075	11.37
9.83	5090	14500	1126	11.64
10.00	5281	15500	1182	11.92

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ROUTING OF PMF THROUGH WEBSTER LAKE

TIME HRS	VOLUME AC-FT	INFLOW CFS	DISCHARGE CFS	HEAD FEET
10.17	5485	16500	1244	12.23
10.33	5701	17500	1311	12.55
10.50	5931	18500	1385	12.90
10.67	6173	19500	1466	13.26
10.83	6427	20500	1554	13.64
11.00	6695	21500	1649	14.04
11.17	6970	21833	1753	14.45
11.33	7243	21500	1862	14.86
11.50	7511	21167	1975	15.27
11.67	7772	20833	2046	15.66
11.83	8028	20500	2098	16.04
12.00	8279	20167	2149	16.37
12.17	8525	19833	2194	16.70
12.33	8765	19500	2238	17.02
12.50	9000	19167	2281	17.33
12.67	9230	18833	2323	17.64
12.83	9455	18500	2364	17.94
13.00	9675	18167	2404	18.23
13.17	9889	17833	2443	18.52
13.33	10099	17500	2482	18.80
13.50	10303	17167	2519	19.07
13.67	10502	16833	2555	19.34
13.83	10696	16500	2591	19.60
14.00	10885	16167	2626	19.85
14.17	11069	15833	2660	20.09
14.33	11248	15500	2692	20.33
14.50	11422	15167	2724	20.56
14.67	11591	14833	2755	20.79
14.83	11755	14500	2786	21.01
15.00	11914	14167	2815	21.22

ROUTING OF PMF THROUGH WEBSTER LAKE

TIME HRS	VOLUME AC-FT	INFLOW CFS	DISCHARGE CFS	HEAD FEET
15.17	12068	13833	2843	21.42
15.33	12217	13500	2871	21.62
15.50	12361	13167	2897	21.81
15.67	12500	12833	2923	22.00
15.83	12634	12500	2948	22.18
16.00	12763	12167	2972	22.35
16.17	12887	11833	2995	22.52
16.33	13006	11500	3017	22.67
16.50	13121	11167	3038	22.83
16.67	13230	10833	3059	22.97
16.83	13335	10500	3078	23.11
17.00	13435	10167	3097	23.25
17.17	13530	9833	3115	23.37
17.33	13620	9500	3132	23.49
17.50	13705	9167	3148	23.61
17.67	13785	8833	3163	23.71
17.83	13861	8500	3178	23.81
18.00	13932	8167	3191	23.91
18.17	13998	7833	3204	24.00
18.33	14060	7500	3216	24.08
18.50	14116	7167	3227	24.16
18.67	14168	6833	3237	24.22
18.83	14215	6500	3246	24.29
19.00	14258	6167	3254	24.34
19.17	14296	5833	3262	24.39
19.33	14329	5500	3269	24.44
19.50	14357	5167	3275	24.48
19.67	14381	4833	3280	24.51
19.83	14400	4500	3284	24.53
20.00	14414	4167	3287	24.55

ROUTING OF PMF THROUGH WEBSTER LAKE

TIME HRS	VOLUME AC-FT	INFLOW CFS	DISCHARGE CFS	HEAD FEET
20.17	14424	3833	3290	24.57
20.33	14429	3500	3292	24.57
20.50	14430	3167	3293	24.57
20.67	14426	2833	3293	24.57
20.83	14417	2500	3292	24.56
21.00	14404	2167	3291	24.54
21.17	14386	1833	3288	24.51
21.33	14364	1500	3285	24.49
21.50	14337	1167	3281	24.45
21.67	14306	833	3276	24.41
21.83	14270	500	3271	24.36
22.00	14229	167	3264	24.31

P. 22-23

DAMS 148 CHANCE POND #9 7-13-79 DWW p. 23 of 39
WEBSTER LAKE

THE RESULTS OF THE ROUTING INDICATE THAT THE OUTFLOW PEAK WOULD OCCUR AT 20.5 HOURS AFTER THE START OF THE STORM AND EQUAL 3300 cfs. THUS THE PEAK INFLOW OF 22,000 cfs HAS BEEN ATTENUATED BY 85%. THE PEAK STAGE OF THE LAKE WOULD BE 24.5 FEET ABOVE THE CULVERT INVERT OR 19 FEET ABOVE THE NORMAL POOL ELEVATION. A CHECK OF THE TOPOGRAPHIC MAPS USED BY ANCO FOR THE FLOOD INSURANCE STUDY INDICATES THAT AN ELEVATION OF 419.5 IS FEASIBLE WITHOUT OVERTOPPING ANY OF THE BANKS SURROUNDING THE LAKE AND DISCHARGING IN AN ALTERNATE DIRECTION. THUS AN PMF MAGNITUDE EVENT WOULD CAUSE SEVERE FLOODING TO THE AREA SURROUNDING WEBSTER LAKE BUT THE PEAK FLOW EXPERIENCED AT THE DAM WOULD BE LIMITED. THIS ASSUMES THAT THE RAILROAD EMBANKMENT COULD WITHSTAND 19' OF HEAD ABOVE NORMAL POND ELEVATION.

DAMS 148 CHANGE POND #9 WEBSTER LAKE 7-13-78 DWW p.24 of 39

A CONTINUITY CHECK WAS PERFORMED TO INSURE THAT THE DIFFERENCE IN AREA BENEATH THE INFLOW + OUTFLOW HYDROGRAPHS EQUALED THE CHANGE IN STORAGE.

TOTAL VOLUME OF INFLOW IN 22 HOURS = 7.128×10^8 ft^3

TOTAL VOLUME OF OUTFLOW IN 22 HOURS = 1.359×10^8 ft^3

FINAL STORAGE (EL=) = 14400 AF
INITIAL STORAGE (EL=) = 1000 AF

CHANGE IN STORAGE = 13400 AF

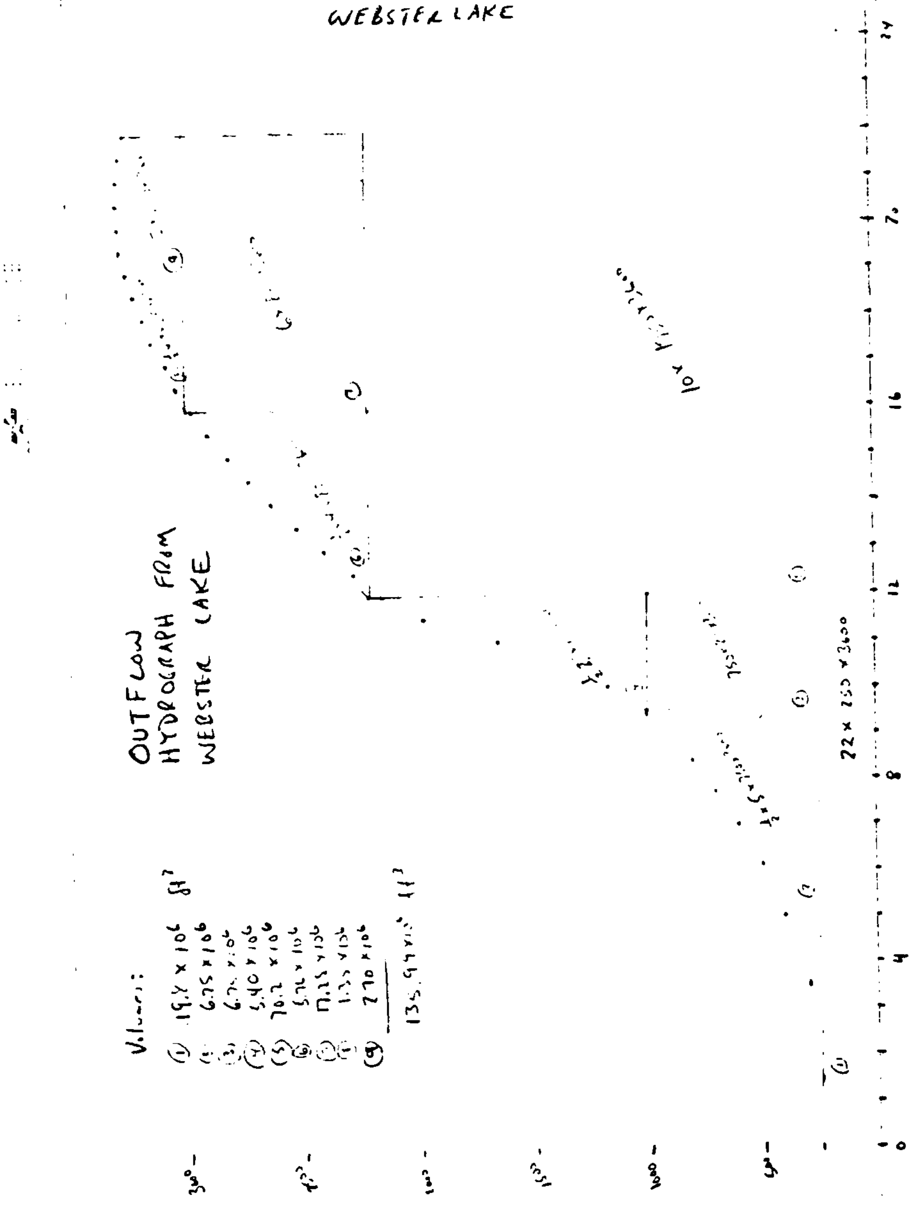
DIFFERENCE IN HYDROGRAPHS = $\frac{7.128 \times 10^8 - 1.359 \times 10^8}{43560}$

= 13240 AF

CONTINUITY ERROR = $\frac{160}{13400}$ = 1.2%

THIS IS ACCEPTABLE

DAMS 148 | CHANCE POND #9 WEBSTER LAKE | 7-13-78 DWW p 25 9A



DAMS 148 | CHANCE POND #9 WEBSTER LAKE | 7-13-79 DWW | p 26 of 39

THE USE OF 3300 CFS AS THE TEST FOR THE CHANCE POND DAM DOES NOT ACCOUNT FOR THE ADDITIONAL DRAINAGE AREA DOWNSTREAM OF THE RR CULVERT AND UPSTREAM OF THE DAM ≈ 2.2 SQMI. A PMF FROM A BASIN OF 2.2 SQMI ALONE WOULD IN ACCORDANCE WITH THE COE CURVES YIELD 1500-2000 CSM OR 3000 → 4000 cfs. THE RUNOFF FROM THE "LOWER BASIN" CAN BE REASONABLY EXPECTED TO PASS OUT OF THE BASIN BEFORE THE PEAK FROM WEBSTER LAKE WOULD OCCUR. BUT GIVEN THAT A TAIL FROM THE LOWER BASIN HYDROGRAPH WOULD STILL BE IN THE RIVER WE HAVE SELECTED TO INCREASE THE WEBSTER LAKE OUTFLOW TO 4000 CFS AT CHANCE POND DAM FOR THE SDF.

SDF = 4000 cfs.

DAMS 148 CHANCE POND #9 7-13-79 DWW p.27 of 39
WEBSTER LAKE

AS PART OF THE FLOOD INSURANCE STUDY ANCO DEVELOPED A STAGE-DISCHARGE CURVE FOR THE CHANCE POND DAM. (ATTACHED)

THE RATING CURVE ASSUMES THAT STOPLOGS ARE IN PLACE TO APPROX. THE SAME ELEVATION AS THE SPILLWAY, AND THAT THE ORAFICE WITH GATE IS WIDE OPEN.

IF THE GATE IS OPEN THE STAGE RESULTING FROM A 4000 CFS DISCHARGE WOULD BE 402.5 FT (MSL).

IF THE GATE WERE FOR SOME REASON COMPLETELY CLOSED THE STAGE REQUIRED WOULD BE RAISED BY ABOUT 0.1 FEET TO 402.6 FT (MSL)

THESE ELEVATIONS REPRESENT 3.8 FT OF FLOW OVER THE SPILLWAY, BUT THIS WOULD NOT OVER TOP THE ABUTMENTS OF THE DAM.

THUS AS LONG AS THE RAILROAD CULVERT HOLDS AND SERVES TO ATTENUATE FLOWS ABOVE THE DAM THE POTENTIAL FOR OVER TOPPING IS VERY SMALL.

Anderson-Nichols & Company, Inc.

Subject [illegible] Rating Curve Calculation

JOB NO. [illegible]

Sheet No. 1 of 1 P. 78 / 39

Date [illegible]

Computed [illegible]

Checked

WEBSTER LAKE DAM

RATING CURVE

Q_{TOTAL} = [illegible] discharge [illegible]

$= C_1 A \sqrt{2gh} + C_2 L h^{3/2} + C L h^{3/2}$

$= 0.6 (15) \sqrt{2 g h}$ + 3 ([illegible]) + 3.4 ([illegible])

(1) Spillway crest length [illegible] when W.S.EL. is above [illegible] ft in [illegible]

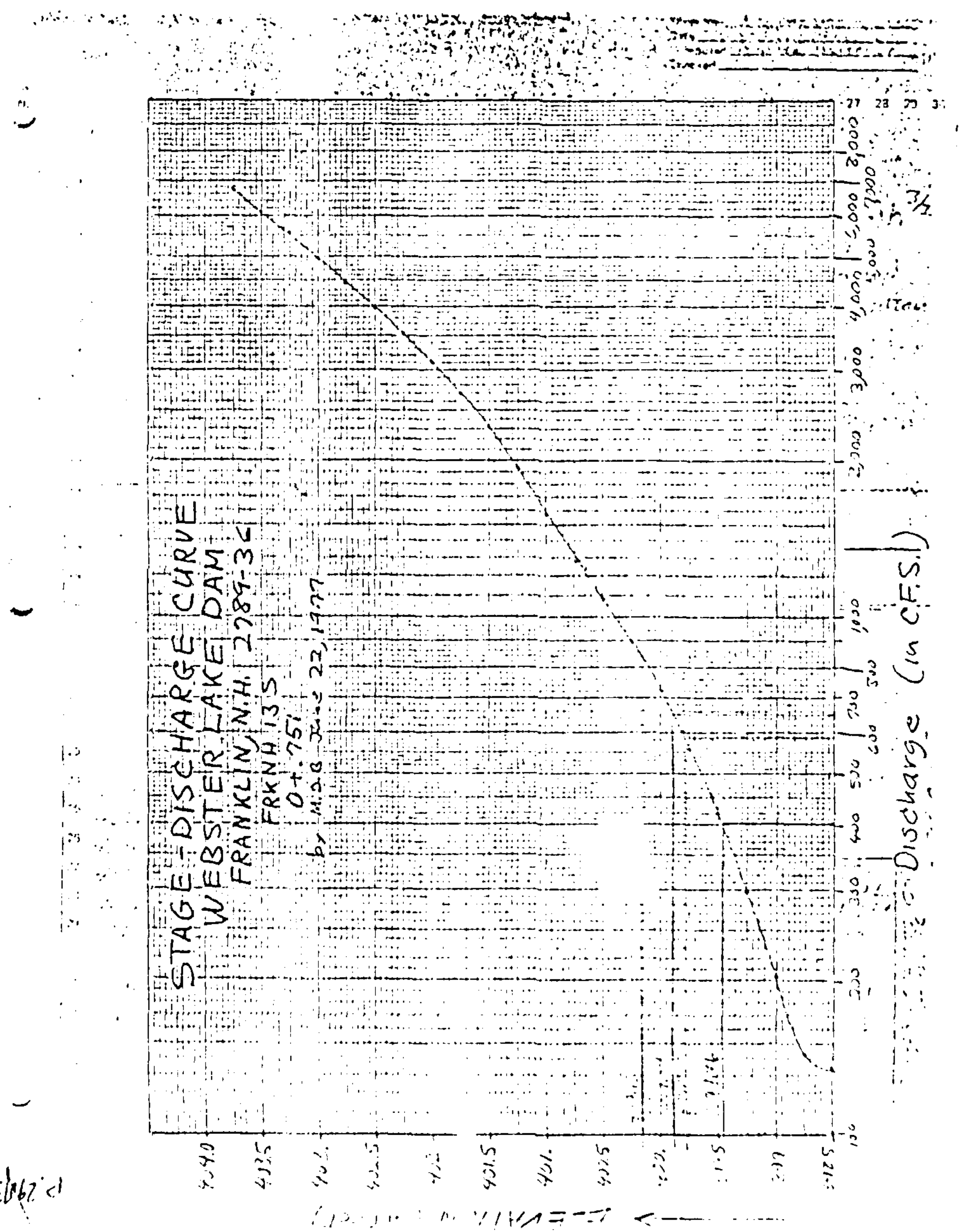
(2) When W.S. EL. is [illegible] ft [illegible] spillway crest [illegible]

h = W.S.EL. − 396.7 (free flowing) h = W.S.EL. − 398.83 (Stoplog spilling) h = W.S.EL. − 398.45 (Crest)

Starting W.S.EL. 398.5

W.S. E.L.	Head: Stoplog	Head: Orifice	Head: Crest	Q (cfs): Stoplog	Q (cfs): Orifice	Q (cfs): Crest	Q (cfs): Total	
398.50	0	4.35	0	0	133.10	0	133.10	
398.75	0	4.60	0.05	0	137.09	3.99	141.08	
399.00	0.17	4.85	0.30	2.11	141.29	58.66	202.15	
399.25	0.42	5.10	0.55	8.21	144.93	115.62	268.81	
399.50	0.67	5.35	0.80	16.53	148.4[illegible]	255.95	420.97	
399.75	0.92	5.60	1.05	26.61	[illegible]	384.11	562.67	
400.00	1.17	5.85	1.30	38.1[illegible]	[illegible]	521.16	722.[illegible]	
400.25	1.42	6.10	1.55	51.02	158.[illegible]	688.12	898.50	
400.50	1.67	6.35	1.80	65.07	161.76	862.14	1066.99	
400.75	1.92	6.60	2.05	80.21	164.1	1047.85	1292.99	Crest New L = 111 Ft
401.00	2.17	6.85	2.30	96.38	168.03	1316.4	1581.8	111
401.25	2.42	7.10	2.55	113.50	171.07	1647.5	1932.1	[illegible]
401.50	2.67	7.35	2.80	131.54	174.05	[illegible]	2428.7	[illegible]
401.75	2.92	[illegible]	3.05	150.44	176.[illegible]	[illegible]	[illegible]	[illegible]
402.00	3.17	7.85	3.30	170.14	179.87	[illegible]	3102	[illegible]
402.25	3.42	8.10	3.55	190.61	182.[illegible]	3161	3534	[illegible]
402.50	3.67	8.35	3.80	211.98	185.51	[illegible]	3994	[illegible]
402.75	3.92	8.60	4.05	234.00	188.[illegible]	4045	4468	[illegible]
403.00	[illegible]	8.85	4.30	239	190.97	4548	4977	[illegible]
403.25	[illegible]	9.10	4.55	252	[illegible]	5082	5527	[illegible]
403.50	[illegible]	9.35	4.80	265	196.[illegible]	[illegible]	6075	[illegible]
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]

[illegible] Crest [illegible]


STAGE-DISCHARGE CURVE
WEBSTER LAKE DAM
Discharge (in CFS.)

STEP 4.

Reach 1:

$Q_{p_1} = 4590$ cfs

$H = 6.9$ ft.

AREA at 6.9 ft = 1030 sq. ft.

$V_1 = L \cdot \text{Area} = \frac{1390(1030)}{43,560} = 32.9$ ac-ft $\leq \frac{1}{2}S$

$Q_{p_{2T}} = Q_{p_1}\left(1 - \frac{32.9}{375}\right) = 4190$ cfs

$H = 6.6$ ft

$V_2 = \frac{1390(970)}{43560} = 30.9$ ac-ft $\leq \frac{1}{2}S$

$V_{ave} = 31.9$ ac-ft

$Q_{p_2} = 4590\left(1 - \frac{31.9}{375}\right) = 4200$ cfs

$H = 6.6$

Reach 2: $Q_{p_1} = 4200$

$H = 12.1$ ft

$V_1 = \frac{1560(760)}{43560} = 27.2$ ac-ft $\leq \frac{1}{2}S$

$Q_{p_{2T}} = Q_{p_1}\left(1 - \frac{27.2}{375}\right) = 3895$

$H = 11.8$ ft

$V_2 = \frac{1560(720)}{43560} = 25.8$ ac-ft $\leq \frac{1}{2}S$

CALCULATION OF DOWNSTREAM DAM FAILURE FLOOD STAGES - BASED ON COE "Rule of Thumb" Guidelines, April 1978.

STEP 1: Reservoir Storage at Time of Failure (Chance Brook Pond only): Assume Failure when water surface is at peak ~~of~~ PMF stage, 3.8 ft over the spillway crest.

$S =$ Normal Storage + Surcharge Storage

$= 23^1 (10')^* + 23(3.8)$

$= 230 + 87$

$= 317$ ac-ft

1. ≈ surface area, Chance Brook Pond

* Assumed depth, Chance Pond.

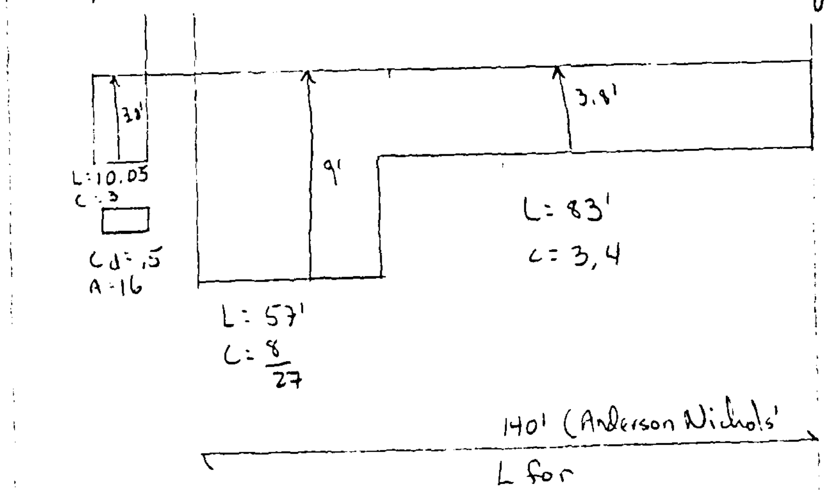
STEP 2: Peak Failure Outflow, Q_P,

$Q_{P_1} = \frac{8}{27} W_b \sqrt{g}\ Y_o^{3/2}$ $W_b = 57'$ (.4×140) $Y_o = 9'$

$= 2544$ cfs

However, the flow over the dam without a breach is 4000 cfs. It seems obvious that dam failure should increase, rather than decrease, the flow. Therefore, we will undertake a more detailed calculation of flow after failure.

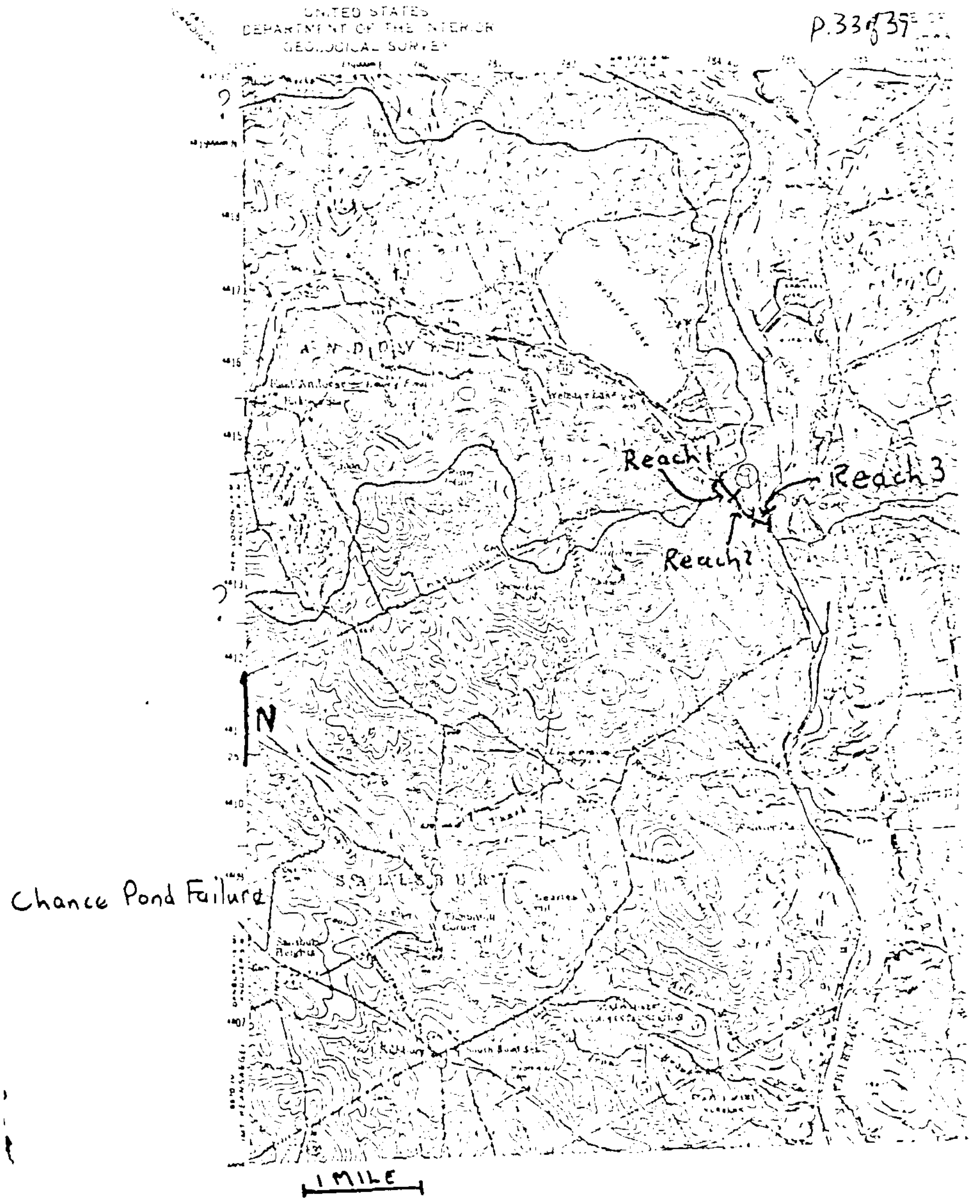
Webster Lake TCG 7-14-78 320/39



$$Q = 3\,(10.05)\,(3.8)^{1.5} + \frac{8\sqrt{g}}{27}(57)\,(9)^{3/2} + 3.4(83)(3.8)^{3/2}$$
$$+ .5\,(16)\,\sqrt{g\cdot 2\cdot 8.35}$$
$$= 223 + 2588 + 2090 + 186 = 5087\,\text{cfs} \sim 5090\,\text{cfs}$$

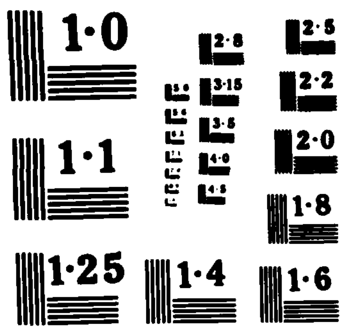
STEP 3: The cross-sections for the downstream reaches shown on the U.S.G.S. Topo Map are plotted on the attached sheet. Computer output tables of the stage-discharge relationships are attached.

The cross-sections used are from the Anderson-Nichols Co. F.I.S. Work.


UNITED STATES
DEPARTMENT OF THE INTERIOR
GEOLOGICAL SURVEY
p.33 of 37
Reach 1
Reach 3
Reach 2
N
Chance Pond Failure
1 MILE

AD-A156 431 NATIONAL PROGRAM FOR INSPECTION OF NON-FEDERAL DAMS CHANCE BROOK DAM (NH..(U) CORPS OF ENGINEERS WALTHAM MA NEW ENGLAND DIV AUG 78 2/2

UNCLASSIFIED F/G 13/13 NL



NATIONAL BUREAU OF STANDARDS
MICROCOPY RESOLUTION TEST CHART

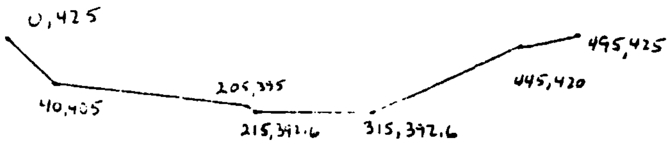
Webster Lake TCC 7-10-78 34/39

Reach 1 - Dam to Kimball St. Bridge

L=1390'

S=.0018

N=.04

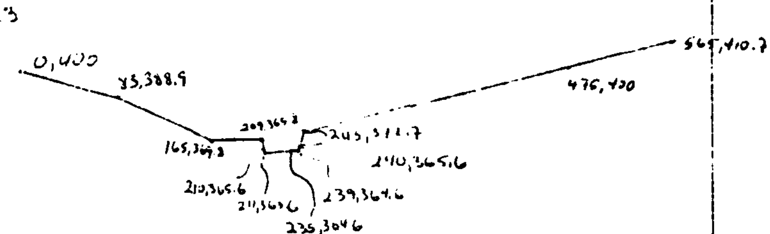


Reach 2 - Kimball St. Bridge to RR Bridge

L=1560'

S=.0023

N=.04

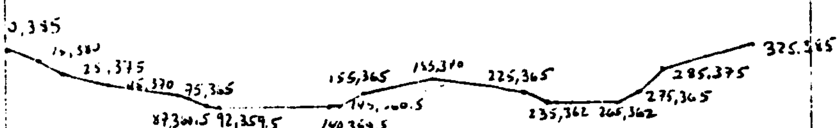


Reach 3 - RR Bridge to Main St. Bridge

L=770'

S=.098

N=.04



DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.0	392.6	0.0	0.0	0.0	0.0	0.0
1.5	394.1	160.0	113.7	1.4	201.0	317.7
3.0	395.6	342.3	134.7	2.5	637.5	1007.6
4.5	397.1	567.4	166.0	3.4	1283.9	2029.0
6.0	398.6	840.3	198.9	4.2	2197.3	3472.5
7.5	400.1	1161.0	231.0	5.0	3408.9	5387.4
9.0	401.6	1529.5	263.0	5.8	4949.0	7821.4
10.5	403.1	1945.8	295.1	6.6	6846.6	10820.3
12.0	404.6	2409.9	327.2	7.4	9129.7	14428.4
13.5	406.1	2913.1	343.5	8.5	12122.8	19158.6
15.0	407.6	3432.5	354.1	9.7	15616.1	24679.5
16.5	409.1	3967.2	364.8	10.9	19489.8	30801.4
18.0	410.6	4517.0	375.4	12.0	23738.5	37516.0
19.5	412.1	5082.0	386.0	13.2	28359.0	44818.1
21.0	413.6	5662.1	396.6	14.3	33349.2	52704.6
22.5	415.1	6257.5	407.3	15.4	38708.5	61174.3
24.0	416.6	6868.0	417.9	16.4	44436.7	70227.1
25.5	418.1	7493.7	428.5	17.5	50534.7	79864.2
27.0	419.6	8134.5	439.1	18.5	57003.6	90087.6
28.5	42[illegible].1	8793.8	455.5	19.3	63347.3	100113.1
30.0	422.6	9479.6	473.9	20.0	69920.8	110501.6
31.5	424.1	10192.4	492.4	20.7	76920.8	121564.3

WEBSTER LAKE DAM - REACH 1

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DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.0	363.6	0.0	0.0	0.0	0.0	0.0
1.5	365.1	26.7	30.4	0.9	24.5	43.7
3.0	366.6	72.0	33.9	2.1	118.9	212.3
4.5	368.1	119.4	37.3	3.2	259.6	463.8
6.0	369.6	169.0	40.7	4.2	437.1	780.9
7.5	371.1	281.4	92.4	3.0	591.2	1056.1
9.0	372.6	413.7	100.9	4.1	1060.3	1894.1
10.5	374.1	564.8	119.5	4.7	1591.4	2843.0
12.0	375.6	744.5	138.8	5.4	2282.0	4076.7
13.5	377.1	952.8	158.2	6.0	3156.1	5638.3
15.0	378.6	1189.7	177.5	6.7	4231.7	7559.8
16.5	380.1	1455.3	196.9	7.4	5526.1	9872.1
18.0	381.6	1749.4	216.2	8.1	7056.0	12605.1
19.5	383.1	2072.2	235.5	8.8	8837.4	15787.5
21.0	384.6	2423.6	254.9	9.5	10886.0	19447.3
22.5	386.1	2803.6	274.2	10.2	13217.0	23611.4
24.0	387.6	3212.2	293.5	10.9	15845.0	28306.3
25.5	389.1	3649.5	313.5	11.6	18760.1	33513.9
27.0	390.6	4119.9	337.6	12.2	21857.6	39047.5
28.5	392.1	4626.0	361.6	12.8	25325.9	45243.5
30.0	393.6	5168.0	385.6	13.4	29182.4	52132.9
31.5	395.1	5745.7	409.7	14.0	33444.3	59746.6
33.0	396.6	6359.2	433.7	14.7	38128.6	68114.8
34.5	398.1	7008.5	457.8	15.3	43252.1	77267.6
36.0	399.6	7693.6	481.8	16.0	48831.2	87234.4

WEBSTER LAKE DAM - REACH 2

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DEPTH	ELEV	AREA	WPER	HYD-R	AR2/3	Q
0.0	359.5	0.0	0.0	0.0	0.0	0.0
1.0	360.5	53.0	58.2	0.9	49.8	580.6
2.0	361.5	113.4	63.5	1.8	167.1	1948.5
3.0	362.5	194.6	102.2	1.9	299.0	3486.1
4.0	363.5	301.5	114.5	2.6	575.1	6706.7
5.0	364.5	419.9	126.7	3.3	933.7	10888.3
6.0	365.5	551.1	143.7	3.8	1351.1	15754.9
7.0	366.5	702.1	165.3	4.2	1842.2	21482.4
8.0	367.5	874.1	187.0	4.7	2445.3	28515.3
9.0	368.5	1067.1	208.6	5.1	3169.9	36964.2
10.0	369.5	1281.1	230.2	5.6	4025.0	46936.2
11.0	370.5	1514.1	243.8	6.2	5118.4	59685.8
12.0	371.5	1754.1	249.4	7.0	6443.9	75142.5
13.0	372.5	1999.1	254.9	7.8	7896.5	92082.1
14.0	373.5	2249.1	260.5	8.6	9473.6	110473.1
15.0	374.5	2504.1	266.0	9.4	11173.2	130291.3
16.0	375.5	2764.3	271.9	10.2	12981.3	151376.6
17.0	376.5	3030.3	278.3	10.9	14898.4	173731.2
18.0	377.5	3302.3	284.7	11.6	16936.6	197499.0
19.0	378.5	3580.3	291.0	12.3	19096.0	222680.5
20.0	379.5	3864.3	297.4	13.0	21376.9	249278.3
21.0	380.5	4154.4	304.2	13.7	23756.7	277029.0
22.0	381.5	4451.4	311.5	14.3	26237.4	305956.9
23.0	382.5	4755.4	318.8	14.9	28843.7	336349.0
24.0	383.5	5066.4	326.1	15.5	31576.8	368220.2
25.0	384.5	5384.4	333.3	16.2	34438.1	401585.8

WEBSTER LAKE DAM - REACH 3

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STEP 4:

Reach 1: $Q_{P_1} = 5090$ cfs

$H = 7.3'$

Area at $7.3' = 1120$ sq.ft.

$$V_1 = L \cdot Area = \frac{1390(1120)}{43560} = 35.7 \text{ ac-ft} \leq \tfrac{1}{2}S$$

$$Q_{P2T} = Q_{P_1}\left(1 - \frac{35.7}{317}\right) = 4520 \text{ cfs}$$

$H = 6.8$ ft

$$V_2 = \frac{1390(1010)}{43,560} = 32.2 \text{ ac-ft}$$

$V_{ave} = 33.95$ ac-ft

$$Q_{P2} = 5090\left(1 - \frac{33.95}{317}\right) = 4540 \text{ cfs}$$

$H = 6.8$

Reach 2: $Q_{P_2} = 4540$

$H = 12.4$ ft

Area at 12.4 ft = 800 ft²

$$V_1 = \frac{1560(800)}{43560} = 28.65 \text{ acres-ft} \leq \tfrac{1}{2}S$$

$$Q_{P2T} = Q_{P_1}\left(1 - \frac{28.65}{317}\right) = 4130$$

$H = 12.1$

$$V_2 = \frac{1560(760)}{43560} = 27.22 \text{ acres-ft} \leq \tfrac{1}{2}S$$

$V_{ave} = 27.94 \text{ ac-ft}$.

$Q_{P2} = 4540\left(1 - \frac{27.94}{317}\right) = 4140 \text{ cfs}$

$H = 12.1$

Reach 3: $Q_{P1} = 4140$

$H = 3.2 \text{ ft}$

$V_1 = \frac{215\,(770)}{43560} = 3.80 \text{ ac-ft} \leq \frac{1}{2} S$

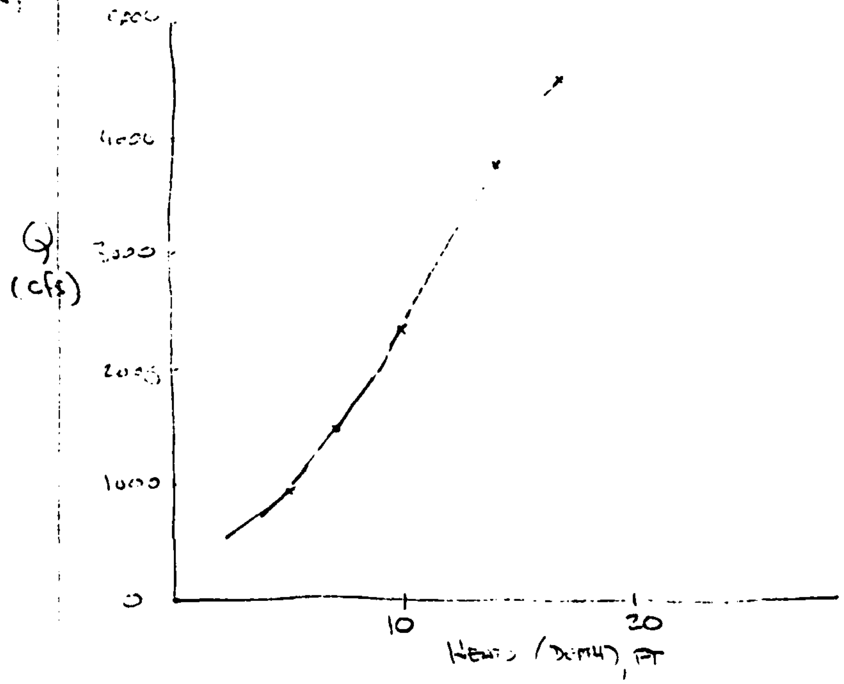
$Q_{P2} = 4140\left(1 - \frac{3.8}{317}\right) = 4090$

$H = 3.2 \text{ ft}$, no attenuation

B & M Railroad Bridge (Basin 2)

17'x14' Culvert — Rating Curve

H/D	H	Q/B	Q
.35	4.9	55	990
.5	7.0	84	1512
.7	9.2	130	2340
1.0	14.0	210	3780
1.2	16.8	250	4500



APPENDIX E

INFORMATION AS CONTAINED IN

THE NATIONAL INVENTORY OF DAMS

INVENTORY OF DAMS IN THE UNITED STATES

STATE	IDENTITY NUMBER	DIVISION	STATE	COUNTY	CONGR. DIST.	STATE	COUNTY	CONGR. DIST.	NAME	LATITUDE (NORTH)	LONGITUDE (WEST)	REPORT DATE DAY MO YR
NH	410	NED	NH	013	02				WEBSTER LAKE DAM	4327.0	7140.2	08SEP78

POPULAR NAME	NAME OF IMPOUNDMENT
CHANCE BROOK DAM	WEBSTER LAKE AND CHANCE POND

REGION	BASIN	RIVER OR STREAM	NEAREST DOWNSTREAM CITY-TOWN-VILLAGE	DIST. FROM DAM (MI.)	POPULATION
01	05	CHANCE POND BROOK	FRANKLIN	1	7292

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUCTURAL HEIGHT (FT.)	HYDRAULIC HEIGHT (FT.)	IMPOUNDING CAPACITIES MAXIMUM (ACRE-FT.)	IMPOUNDING CAPACITIES NORMAL (ACRE-FT.)	DIST	OWN	FED R	PRV/FED	SCS A	VER/DATE
CTPG	1873	R	15	14	2650	1100	NED	N	N	N	N	17AUG78

REMARKS

D/S HAS	SPILLWAY CREST LENGTH	SPILLWAY TYPE	SPILLWAY WIDTH (FT.)	MAXIMUM DISCHARGE (FT.)	VOLUME OF DAM (CY)	POWER CAPACITY INSTALLED (MW)	POWER CAPACITY PROPOSED (MW)	NAVIGATION LOCKS NO.	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)	LENGTH (FT.)	WIDTH (FT.)
1	133	C	110	700	400											

OWNER	ENGINEERING BY	CONSTRUCTION BY
NH WATER RES BD	NH WATER RES BD	NH WATER RES BD

REGULATORY AGENCY			
DESIGN	CONSTRUCTION	OPERATION	MAINTENANCE
NONE	NONE	NONE	NONE

INSPECTION BY	INSPECTION DATE DAY MO YR	AUTHORITY FOR INSPECTION
GOLDBERG ZOINO DUNNICLIFF + ASSOC	01JUN78	PL 92-367

REMARKS

END

FILMED

8-85

DTIC